Imperial Solar Energy Center South

Appendix D

Geotechnical Investigative Report

Prepared by Landmark Consultants, Inc.

May 2010

Geotechnical Investigation Report

Imperial Solar Energy Center South

Pulliam Road and Anza Road
Calexico, California

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Geotechnical Investigation
Proposed Imperial Solar Energy Center South
Pulliam Road and Anza Road
Calexico, California
LCI Report No. LE10094

Dear Mr. Johnson:

This geotechnical report is provided for design and construction of the proposed Imperial Solar Energy Center South project located south of State Route 98 around the intersection of Pulliam Road and Anza Road west of Calexico, California. Our geotechnical investigation was conducted in response to your request for our services. The enclosed report describes our soil engineering investigation and presents our professional opinions regarding geotechnical conditions at the site to be considered in the design and construction of the project.

This executive summary presents ***selected*** elements of our findings and recommendations only. This summary ***does not*** present all details needed for the proper application of our findings and recommendations. Our findings, recommendations, and application options are related ***only through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them. The findings of this study are summarized below:

- Clay soils (CL-CH) of high to very high expansion predominate the site.

- Foundation designs for buildings shall mitigate expansive soil conditions by one of the following methods:

    1. Remove and replace upper 3.0 feet of clay soils with non-expansive sands.
    2. Design foundations to resist expansive forces in accordance with the 2007 California Building Code (CBC) Chapter 18, Section 1805 or the Post-Tensioning Institute, 2004 method. This requires grade-beam stiffened of floor slabs (18 feet maximum on center) or post tensioned floor slabs. Design soil bearing pressure = 1,500 psf.

- The risk of liquefaction induced settlement is moderate to high (estimated settlement of 1 to 4.5 inches at 18 to 48 feet below ground surface.

- The clay soils are aggressive to concrete and steel. Concrete mixes shall have a maximum water cement ratio of 0.45 and a minimum compressive strength of 4,500 psi (minimum of 6 sacks Type II/V cement per cubic yard).

- All reinforcing bars, anchor bolts and hold downs shall have a minimum concrete cover of 3.0 inches. No hold down straps are allowed at the foundation perimeter.

- The clay soils are non-absorptive and poor for onsite sewage disposal systems or for infiltration in stormwater basins, except for the northwestern portion of the site which has sandy near-surface soils (B-12, B-14 and B-15 locations).

We did not encounter soil conditions that would preclude development of the proposed project provided the recommendations contained in this report are implemented in the design and construction of this project.

We appreciate the opportunity to provide our findings and professional opinions regarding geotechnical conditions at the site. If you have any questions or comments regarding our findings, please call our office at (760) 370-3000.

Respectfully Submitted,
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Section 1
INTRODUCTION

1.1 Project Description

This report presents the findings of our geotechnical investigation for the proposed Imperial Solar Energy Center South project located south of State Route 98 around the intersection of Pulliam Road and Anza Road approximately 7 miles west of Calexico, California (See Vicinity Map, Plate A-1). The proposed project will consist of approximately 903 acres of PV solar panels mounted on steel racks supported by short piers, shallow driven piles or shallow spread footings. Also, the proposed solar energy facility will have maintenance/storage building(s), inverter stations, and an electrical substation. The photovoltaic modules will be ground mounted on single axis trackers or fixed mount structures. A site plan for the proposed development was not made available to us at the time that this report was prepared.

The small office and maintenance/storage building is planned to consist of slab-on-grade foundation with steel frame and/or wood-frame construction. Footing loads at exterior bearing walls are estimated at 1 to 5 kips per lineal foot. Column loads are estimated to range from 5 to 30 kips. If structural loads exceed those stated above, we should be notified so we may evaluate their impact on foundation settlement and bearing capacity. Site development will include minimal site grading, building pad preparation, septic system installation, underground utility installation, and site paving at the O & M building.

1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 50 feet of subsurface soil at selected locations within the site for evaluation of physical/engineering properties. From study of field and laboratory data, professional opinions were developed and are provided in this report regarding geotechnical conditions at this site and the effect on design and construction.

The scope of our services consisted of the following:

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected samples.
- Review of the available literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.
- Preparation of this report presenting our findings, professional opinions, and recommendations for the geotechnical aspects of project design and construction.

This report addresses the following geotechnical issues:

- Subsurface soil and groundwater conditions
- Site geology, regional faulting and seismicity, near source factors, and site seismic accelerations
- Liquefaction potential and its mitigation
- Expansive soil and methods of mitigation
- Aggressive soil conditions to metals and concrete

Professional opinions with regard to the above issues are presented for the following:

- Site grading and earthwork
- Building pad and foundation subgrade preparation
- Allowable soil bearing pressures and expected settlements
- Typical capacities for drilled piers and driven steel piles
- Concrete slabs-on-grade
- Excavation conditions and buried utility installations
- Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
- Seismic design parameters

Our scope of work for this report did not include an evaluation of the site for the presence of environmentally hazardous materials or conditions, groundwater mounding (due to applied water to site), or landscape suitability of the soil.

1.3 Authorization

Authorization to proceed with our work was provided by signed agreement with Tenaska on April 20, 2010. We conducted our work according to our written proposal dated April 2, 2010.

Section 2
METHODS OF INVESTIGATION

2.1 Field Exploration

Subsurface exploration was performed on April 29 and 30, 2010 using 2R Drilling of Ontario, California to advance fifteen (15) borings to depths of 20 to 50 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 55 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernable site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A professional engineer observed the drilling operations and maintained logs of the soil encountered with sampling depths. During drilling soils were visually classified according to the Unified Soil Classification System and relatively undisturbed and bulk samples of the subsurface materials were obtained at selected intervals. The relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler. In addition, Standard Penetration Tests (SPT) was performed in accordance with ASTM D1586. The samples were obtained by driving the samplers ahead of the auger tip at selected depths using a 140-pound CME automatic hammer with a 30-inch drop. The number of blows required to drive the samplers the last 12 inches of an 18-inch drive depth into the soil is recorded on the boring logs as "blows per foot". Blow counts (N values) reported on the boring logs represent the field blow counts. No corrections have been applied for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter. Pocket penetrometer readings were also obtained to evaluate the stiffness of cohesive soils retrieved from sampler barrels.

After logging and sampling the soil, the exploratory borings were backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill.

The subsurface logs are presented on Plates B-1 through B-15 in Appendix B. A key to the log symbols is presented on Plate B-16. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

2.2 Laboratory Testing

Laboratory tests were conducted on selected bulk (auger cuttings) and relatively undisturbed soil samples obtained from the soil borings to aid in classification and evaluation of selected engineering properties of the site soils. The tests were conducted in general conformance to the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory testing program consisted of the following tests:

- Plasticity Index (ASTM D4318) – used for soil classification and expansive soil design criteria
- Particle Size Analyses (ASTM D422) – used for soil classification and liquefaction evaluation
- Unit Dry Densities (ASTM D2937) and Moisture Contents (ASTM D2216) – used for insitu soil parameters
- Direct Shear (ASTM D3080) – used for soil strength determination
- Unconfined Compression (ASTM D2166) – used for soil strength estimates.
- Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods) – used for concrete mix proportions and corrosion protection requirements.

The laboratory test results are presented on the subsurface logs (Appendix B) and on Plates C-1 through C-13 in Appendix C.

Engineering parameters of soil strength, compressibility and relative density utilized for developing design criteria provided within this report were obtained from the field and laboratory testing program.

Section 3
DISCUSSION

3.1 Site Conditions

The project site is located south of State Route 98 around the intersection of Pulliam Road and Anza Road approximately 7 miles west of Calexico, California. A majority of the project site is roughly rectangular in plan view and consists of approximately 903 acres that are currently agricultural fields. There is an approximately 160 acre portion (agricultural field) located west of the Westside Main Canal at the northwest corner of Anza and Pulliam Roads that extends to the north. The agricultural fields are currently in crop production. The All American Canal abuts the southeastern boundary of the project site. The West Side Main Canal bisects the site in a north-south direction. Adjacent properties are flat-lying and are approximately at the same elevation with this site, consisting of agricultural fields.

The project site lies at an elevation of approximately 5 feet above to 15 feet below mean sea level (MSL) (El. 1005 to 985 local datum) in the Imperial Valley region of the California low desert. The surrounding properties lie on terrain which is flat (planar), part of a large agricultural valley, which was previously an ancient lake bed covered with fresh water to an elevation of 43± feet above MSL. Annual rainfall in this arid region is less than 3 inches per year with four months of average summertime temperatures above 100 °F. Winter temperatures are mild, seldom reaching freezing.

3.2 Geologic Setting

The project site is located in the Imperial Valley portion of the Salton Trough physiographic province. The Salton Trough is a topographic and geologic structural depression resulting from large scale regional faulting. The trough is bounded on the northeast by the San Andreas Fault and Chocolate Mountains and the southwest by the Peninsular Range and faults of the San Jacinto Fault Zone. The Salton Trough represents the northward extension of the Gulf of California, containing both marine and non-marine sediments since the Miocene Epoch. Tectonic activity that formed the trough continues at a high rate as evidenced by deformed young sedimentary deposits and high levels of seismicity. Figure 1 shows the location of the site in relation to regional faults and physiographic features.

The Imperial Valley is directly underlain by lacustrine deposits, which consist of interbedded lenticular and tabular silt, sand, and clay. The Late Pleistocene to Holocene lake deposits are probably less than 100 feet thick and derived from periodic flooding of the Colorado River which intermittently formed a fresh water lake (Lake Cahuilla). Older deposits consist of Miocene to Pleistocene non-marine and marine sediments deposited during intrusions of the Gulf of California. Basement rock consisting of Mesozoic granite and Paleozoic metamorphic rocks are estimated to exist at depths between 15,000 - 20,000 feet.

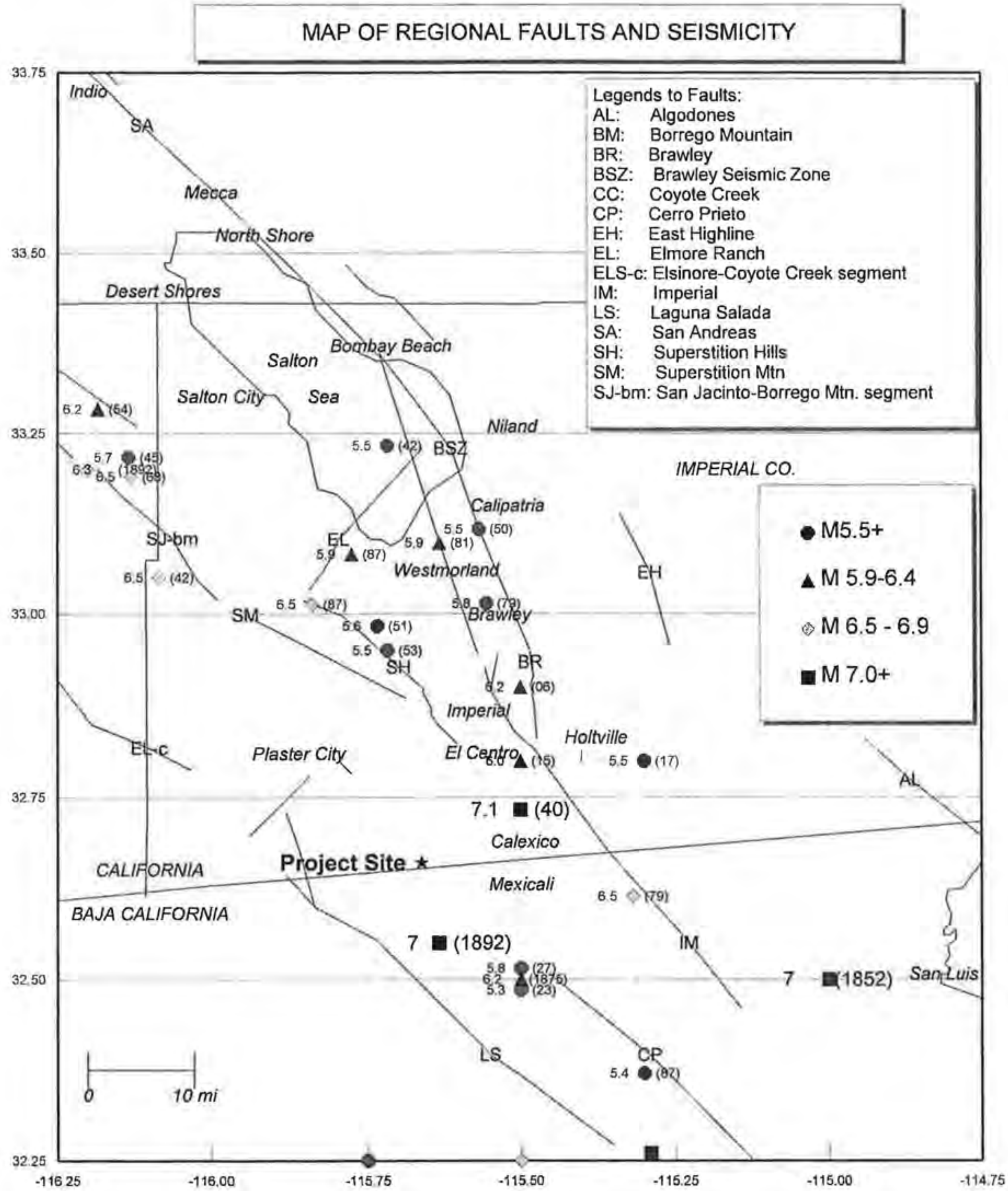
3.3 Seismicity and Faulting

Faulting and Seismic Sources: We have performed a computer-aided search of known faults or seismic zones that lie within a 62 mile (100 kilometer) radius of the project site as shown on Figure 1 and Table 1. The search identifies known faults within this distance and computes deterministic ground accelerations at the site based on the maximum credible earthquake expected on each of the faults and the distance from the fault to the site. The Maximum Magnitude Earthquake (Mmax) listed was taken from published geologic information available for each fault (Cao, et. al., 2003 and Jennings, 1994).

Seismic Risk: The project site is located in the seismically active Imperial Valley of southern California and is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. The proposed site structures should be designed in accordance with the 2007 California Building Code (CBC) for a "Maximum Considered Earthquake" (MCE) and with the appropriate site coefficients. The MCE is defined as the ground motion having a 2 percent probability of being exceeded in 50 years.

Seismic Hazards.

- **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the Imperial, Laguna Salada, and Superstition Hills Faults. A further discussion of groundshaking follows in Section 3.4.
- **Surface Rupture.** The project site does not lie within a State of California, Alquist-Priolo Earthquake Fault Zone. Surface fault rupture is considered to be unlikely at the project site because of the well-delineated fault lines through the Imperial Valley as shown on USGS and CGS maps.



Faults and Seismic Zones from Jennings (1994), Earthquakes modified from Ellsworth (1990) catalog.

Figure 1. Map of Regional Faults and Seismicity

Table 1
FAULT PARAMETERS & DETERMINISTIC
ESTIMATES OF PEAK GROUND ACCELERATION (PGA)

Fault Name or Seismic Zone	Distance (mi) & Direction from Site		Fault Type		Fault Length (km)	Maximum Magnitude Mmax (Mw)	Avg Slip Rate (mm/yr)	Avg Return Period (yrs)	Date of Last Rupture (year)	Largest Historic Event >5.5M	(year)	Est. Site PGA (g)
Reference Notes: (1)			(2)	(3)	(2)	(4)	(3)	(3)	(3)	(5)		(6)
Imperial Valley Faults												
Imperial	15	NE	A	B	62	**7.0**	20	79	1979	7.0	1940	**0.19**
Brawley	16	NE	B	B	14	**7.0**	20	---	1979	5.8	1979	**0.18**
Cerro Prieto	16	SE	A	B	116	**7.2**	34	50	1980	7.1	1934	**0.20**
Brawley Seismic Zone	20	NNE	B	B	42	**6.4**	25	24		5.9	1981	**0.11**
East Highline Canal	31	NE	C	C	22	**6.3**	1	774				**0.07**
San Jacinto Fault System												
- Superstition Hills	12	NNE	B	A	22	**6.6**	4	250	1987	6.5	1987	**0.19**
- Superstition Mtn.	16	N	B	A	23	**6.6**	5	500	1440 +/-			**0.15**
- Elmore Ranch	28	NNW	B	A	29	**6.6**	1	225	1987	5.9	1987	**0.10**
- Borrego Mtn	31	NW	B	A	29	**6.6**	4	175		6.5	1942	**0.09**
- Anza Segment	49	NW	A	A	90	**7.2**	12	250	1918	6.8	1918	**0.09**
- Coyote Creek	50	NW	B	A	40	**6.8**	4	175	1968	6.5	1968	**0.07**
- Hot Spgs-Buck Ridge	65	NW	B	A	70	**6.5**	2	354		6.3	1937	**0.05**
- Whole Zone	16	N	A	A	245	**7.5**	---	---				**0.24**
Elsinore Fault System												
- Laguna Salada	8.5	SW	B	B	67	**7.0**	3.5	336		7.0	1891	**0.29**
- Coyote Segment	23	WNW	B	A	38	**6.8**	4	625				**0.13**
- Julian Segment	50	WNW	A	A	75	**7.1**	5	340				**0.08**
- Earthquake Valley	53	WNW	B	A	20	**6.5**	2	351				**0.06**
- Whole Zone	23	WNW	A	A	250	**7.5**	---	---				**0.18**
San Andreas Fault System												
- Coachella Valley	48	N	A	A	95	**7.4**	25	220	1690+/-	6.5	1948	**0.10**
- Whole S. Calif. Zone	48	N	A	A	458	**7.9**	---	---	1857	7.8	1857	**0.13**

Notes:

1. Jennings (1994) and CDMG (1996)
2. CDMG (1996), where Type A faults -- slip rate >5 mm/yr and well constrained paleoseismic data
   Type B faults -- all other faults.
3. WGCEP (1995)
4. CDMG (1996) based on Wells & Coppersmith (1994)
5. Ellsworth Catalog in USGS PP 1515 (1990) and USBR (1976), Mw = moment magnitude,
6. The deterministic estimates of the Site PGA are based on the attenuation relationship of:
   Boore, Joyner, Fumal (1997)

Ground failures were noted along the embankments of the All American Canal and West Side Main Canal after the April 4, 2010 magnitude $7.2M_w$ El Mayor-Cucapah earthquake.

► **Liquefaction.** Liquefaction is a potential design consideration because of underlying saturated sandy substrata. The potential for liquefaction at the site is discussed in more detail in Section 3.7.

Other Secondary Hazards.

► **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps of the region and no indications of landslides were observed during our site investigation.

► **Volcanic hazards.** The site is not located in proximity to any known volcanically active area and the risk of volcanic hazards is considered very low.

► **Tsunamis, seiches, and flooding.** The site does not lie near any large bodies of water, so the threat of tsunami, seiches, or other seismically-induced flooding is unlikely. The water levels in the canals (All American and West Side Main) are at or slightly below the site elevation.

► **Expansive soil.** In general, much of the near surface soils in the Imperial Valley consist of silty clays and clays which are moderate to highly expansive. The expansive soil conditions at this site are discussed in more detail in Section 3.5.

3.4 Site Acceleration and CBC Seismic Coefficients

Site Acceleration: Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Accelerations also are dependent upon attenuation by rock and soil deposits, direction of rupture and type of fault; therefore, ground motions may vary considerably in the same general area. Deterministic horizontal peak ground accelerations (PGA) from maximum probable earthquakes on regional faults have been estimated and are included in Table 1. The deterministic PGA estimate for the project site is based on the ground motion having a 10% probability of being exceeded in 50 years (return period of 475 years).

The computer program FRISKSP (Blake, 2000) was used to obtain the probabilistic and deterministic estimates of the site PGA using the attenuation relationship NEHRP D 250 of Boore, Joyner, and Fumal (1997). The PGA estimate for the Design Basis Earthquake (DBE), defined as an event having a 10% probability of being exceeded in 50 years (return period of 475 years) was estimated to be **0.43g**. The PGA estimate for the Maximum Considered Earthquake (MCE), which is defined as an event having a 2% probability of being exceeded in 50 years (return period of 2,500 years), was estimated to be **0.63g**.

2007 CBC (2006 IBC) Seismic Response Parameters: The 2007 California Building Code (CBC) seismic parameters are based on the Maximum Considered Earthquake for a ground motion with a 2% probability of occurrence in 50 years. This follows the methodology of the 2006 International Building Code (IBC). Table 2 lists the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters given in Chapter 16 of the CBC. The site soils have been classified as Site Class D (stiff soil profile). Design earthquake ground motions are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding MCE ground motions. Design earthquake ground motion data are provided in Table 2.

A site-specific ground motion hazard analysis was prepared in accordance with the 2007 CBC Section 1614A.1.2 (Table 3 and Figure 2). The determination of the site specific ground motions was performed in conformance with the guidelines outlined in ASCE 7-05 Section 21 (21.2.1, 21.2.2, and 21.3).

A ground motion value of **0.37g** (40% of the S_{DS} or $S_{DS}/2.5$) was determined for liquefaction and seismic settlement analysis in accordance with California Geological Survey Note 48. The parameter S_{DS} is derived from the site-specific seismic hazard analysis (ASCE 7-05 Section 21.3) and taken as the spectral acceleration at a period of 0.2 seconds.

3.5 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on April 29 and 30, 2010 consist of dominantly of clays with interbedded silts and sandy silts. The subsurface logs (Plates B-1 through B-15) depict the stratigraphic relationships of the various soil types.

Table 2
2007 California Building Code (CBC) and ASCE 7-05 Seismic Parameters

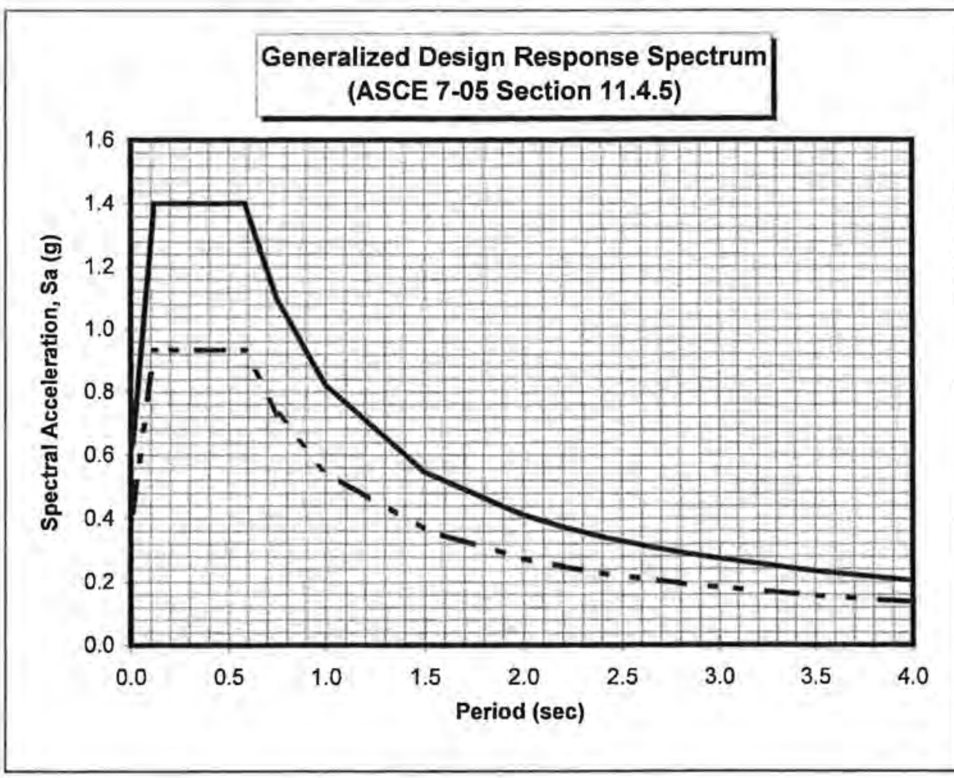
		IBC Reference
Site Class:	**D**	Table 1613.5.2
Latitude:	32.6607 N	
Longitude:	-115.6617 W	

Maximum Considered Earthquake (MCE) Ground Motion

Short Period Spectral Response	S_s	1.40 g	Figure 1613.5(3)
1 second Spectral Response	S_1	0.55 g	Figure 1613.5(4)
Site Coefficient	F_a	1.00	Table 1613.5.3 (1)
Site Coefficient	F_v	1.50	Table 1613.5.3 (2)
Adjusted Short Period Spectral Response	S_{MS}	1.40 g	$= F_a * S_s$
Adjusted 1 second Spectral Response	S_{M1}	0.82 g	$= F_v * S_1$

Design Earthquake Ground Motion

Short Period Spectral Response	S_{DS}	0.93 g	$= 2/3*S_{MS}$
1 second Spectral Response	S_{D1}	0.55 g	$= 2/3*S_{M1}$
	To	0.12 sec	$=0.2*S_{D1}/S_{DS}$
	Ts	0.59 sec	$=S_{D1}/S_{DS}$



2007 CBC

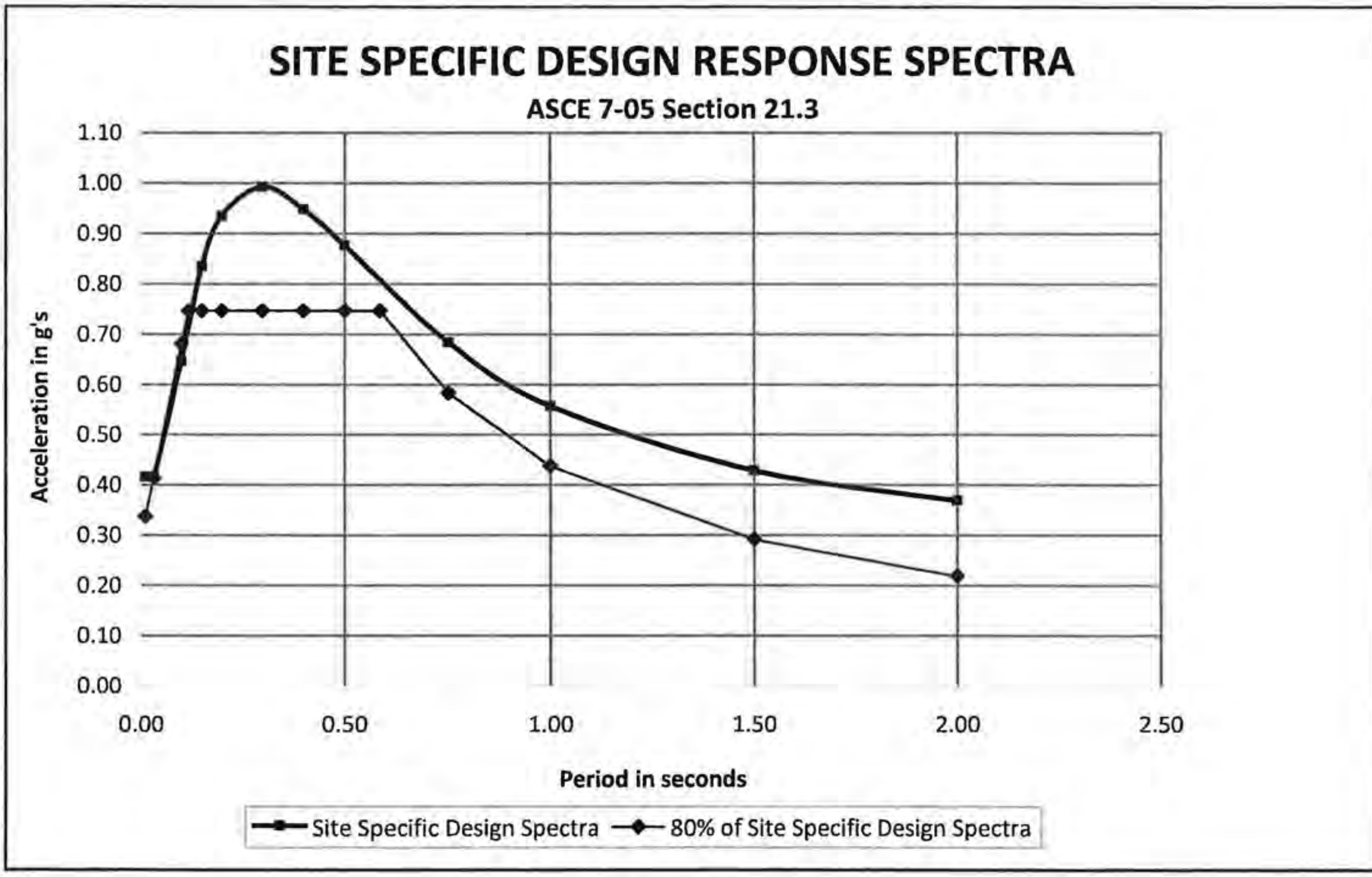
Period T (sec)	Sa (g)	MCE Sa (g)	80% Sa (g)
0.01	0.42	0.63	0.34
0.03	0.52	0.77	0.41
0.10	0.85	1.28	0.68
0.12	0.93	1.40	0.75
0.15	0.93	1.40	0.75
0.20	0.93	1.40	0.75
0.30	0.93	1.40	0.75
0.40	0.93	1.40	0.75
0.50	0.93	1.40	0.75
0.59	0.93	1.40	0.75
0.75	0.73	1.09	0.58
1.00	0.55	0.82	0.44
1.50	0.36	0.55	0.29
2.00	0.27	0.41	0.22
2.20	0.25	0.37	0.20
2.40	0.23	0.34	0.18
2.60	0.21	0.32	0.17
2.80	0.20	0.29	0.16
3.00	0.18	0.27	0.15
3.50	0.16	0.23	0.13
4.00	0.14	0.21	0.11

— · · — · · — Design Response Spectra
——————— MCE Response Spectra

SITE SPECIFIC GROUND MOTION
Table 3

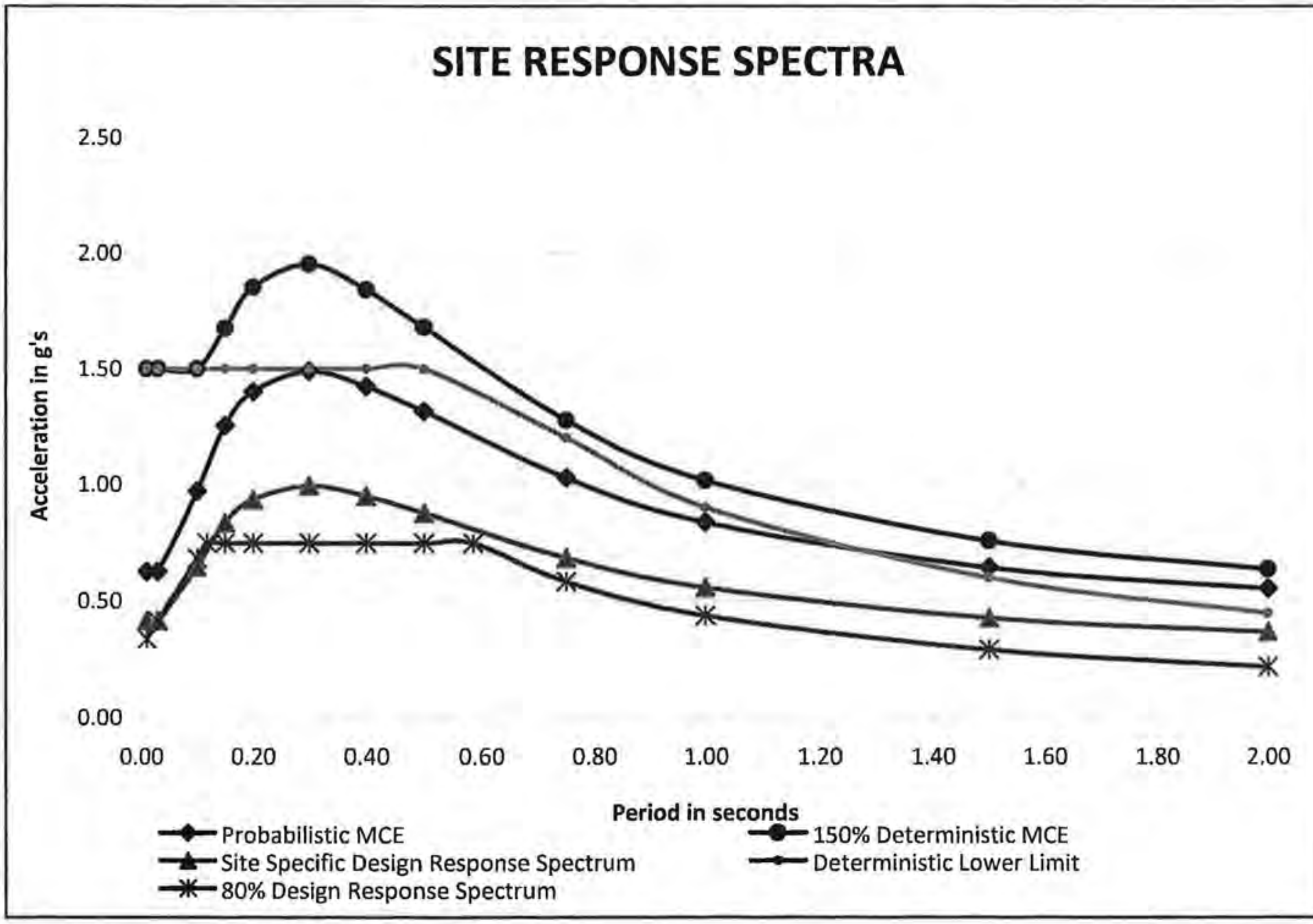
PSHA MCE 2% in 50 years		DETERMINISTIC MCE			DETERMINISTIC LOWER LIMIT		DESIGN RESPONSE SPECTRUM	
Period sec	S_{aM} g's	Period sec	S_{aM} g's	150%S_{aM} g's	Period sec	S_{aM} g's	Period sec	S_a g's
0.01	0.62	0.01	0.53	1.50	0.01	1.50	0.01	0.42
0.03	0.62	0.03	0.53	1.50	0.03	1.50	0.03	0.42
0.10	0.97	0.10	0.87	1.50	0.10	1.50	0.10	0.65
0.15	1.25	0.15	1.12	1.67	0.15	1.50	0.15	0.84
0.20	1.40	0.20	1.23	1.85	0.20	1.50	0.20	0.93
0.30	1.49	0.30	1.30	1.95	0.30	1.50	0.30	0.99
0.40	1.42	0.40	1.23	1.84	0.40	1.50	0.40	0.95
0.50	1.32	0.50	1.12	1.68	0.50	1.50	0.50	0.88
0.75	1.03	0.75	0.85	1.28	0.75	1.20	0.75	0.68
1.00	0.84	1.00	0.68	1.02	1.00	0.90	1.00	0.56
1.50	0.64	1.50	0.50	0.76	1.50	0.60	1.50	0.43
2.00	0.55	2.00	0.42	0.64	2.00	0.45	2.00	0.37

Figure 2. Site specific design response spectra



SITE SPECIFIC DESIGN ACCELERATION PARAMETERS (ASCE 7-05 Section 21.4)			
Short Period Spectral Response (S_{DS})	**0.93**	Adjusted Short Period Spectral Response (S_{MS})	**1.40**
1 second Spectral Response (S_{D1})	**0.74**	Adjusted 1 second Spectral Response (S_{M1})	**1.11**

Figure 3. Ground motion hazard analysis



REFERENCES

PROBABILISTIC MCE	2% in 50 years (2,500 year Return Period) ASCE 7-05 Section 21.2.1
150% DETERMINISTIC MCE	150% Sam of Maximum Considered Earthquake ASCE 7-05 Section 21.2.2
DETERMINISTIC LOWER LIMIT	Lower Limit Deterministic ASCE 7-05 Section 21.2.2
DESIGN RESPONSE SPECTRUM	ASCE 7-05 Section 21.2.3 and 21.3
80% GENERAL PROCEDURE SPECTRUM	80% Sa 2007 CBC and ASCE 7-05 Section 21.3

The native surface clays exhibit high to very high swell potential (Expansion Index, EI = 100 to 160) when tested according to Uniform Building Code Standard 18-2 methods. The clay is expansive when wetted and can shrink with moisture loss (drying). Development of building foundations, concrete flatwork, and asphaltic concrete pavements should include provisions for mitigating potential swelling forces and reduction in soil strength, which can occur from saturation of the soil. Causes for soil saturation include landscape irrigation, broken utility lines, or capillary rise in moisture upon sealing the ground surface to evaporation. Moisture losses can occur with lack of landscape watering, close proximity of structures to downslopes and root system moisture extraction from deep rooted shrubs and trees placed near the foundations. Typical measures used for light industrial projects to remediate expansive soil include:

- replacement of expansive silts/clays with non-expansive sands or silts,
- moisture conditioning subgrade soils to a minimum of 5% above optimum moisture (ASTM D1557) within the drying zone of surface soils,
- design of foundations that are resistant to shrink/swell forces of silt/clay soil.

3.6 Groundwater

Groundwater was encountered in the borings at about 6 to 18 feet during the time of exploration, but may rise with time to approximately 5 to 8 feet below ground surface at this site (see Boring Logs – Plates B-1 thru B15). There is uncertainty in the accuracy of short-term water level measurements, particularly in fine-grained soil. Groundwater levels may fluctuate with precipitation, irrigation of adjacent properties, site landscape watering, drainage, and site grading. The referenced groundwater level should not be interpreted to represent an accurate or permanent condition.

3.7 Liquefaction

Liquefaction occurs when granular soil below the water table is subjected to vibratory motions, such as produced by earthquakes. With strong ground shaking, an increase in pore water pressure develops as the soil tends to reduce in volume. If the increase in pore water pressure is sufficient to reduce the vertical effective stress (suspending the soil particles in water), the soil strength decreases and the soil behaves as a liquid (similar to quicksand).

Liquefaction can produce excessive settlement, ground rupture, lateral spreading, or failure of shallow bearing foundations. Four conditions are generally required for liquefaction to occur:

(1) the soil must be saturated (relatively shallow groundwater);
(2) the soil must be loosely packed (low to medium relative density);
(3) the soil must be relatively cohesionless (not clayey); and
(4) groundshaking of sufficient intensity must occur to function as a trigger mechanism.

All of these conditions exist to some degree at this site.

Methods of Analysis: Liquefaction potential at the project site was evaluated using the 1997 NCEER Liquefaction Workshop methods. The 1997 NCEER methods utilize direct SPT blow counts or CPT cone readings from site exploration and earthquake magnitude/PGA estimates from the seismic hazard analysis. The resistance to liquefaction is plotted on a chart of cyclic shear stress ratio (CSR) versus a corrected blow count $N_{1(60)}$ or Qc_{1N}. A ground acceleration of 0.37g was used in the analysis with an 8-foot groundwater depth.

Liquefaction induced settlements have been estimated using the 1987 Tokimatsu and Seed method. The fine content of liquefiable sands and silts increases the liquefaction resistance in that more ground motion cycles are required to fully develop increased pore pressures. Prior to calculating the settlements, the field SPT blow counts were corrected to account for the type of hammer, borehole diameter, overburden pressure and rod length $N_{1(60)}$ in accordance with Robertson and Wride (1997).

The soil encountered at the points of exploration included saturated silts and silty sands that could liquefy during a CBC Design Basis Earthquake. Liquefaction can occur within silt and sand layers between depths of 18 to 36 feet and 44 to 48 feet. The likely triggering mechanism for liquefaction appears to be strong groundshaking associated with the rupture of the Imperial Fault, Laguna Salada Fault, and possibly the Cerro Prieto Fault.

The analysis is summarized in the table below.

SUMMARY OF LIQUEFACTION ANALYSES

Boring Location	Depth To First Liquefiable Zone (ft)	Potential Induced Settlement (in)
B-2	18	3¾
B-7	18	3¼
B-10	44	1
B-11	18	4½

Liquefaction Induced Settlements: ***Based on empirical relationships, total induced settlements are estimated to be about 1 to 4½ inches should liquefaction occur.*** The magnitude of potential liquefaction induced differential settlement is estimated at be two-thirds of the total potential settlement in accordance with California Special Publication 117; therefore, there is a potential for ¾ to 3 inches of liquefaction induced differential settlement at the project site.

Liquefaction Induced Ground Failure: Based on research from Ishihara (1985) and Youd and Garris (1995) ground rupture or sand boil formation is possible because of the underlying potentially liquefiable soil. Sand boils are conical piles of sand derived from the upward flow of groundwater caused by excess porewater pressures created during strong ground shaking.

Sand boils are not inherently damaging by themselves, but are an indication that liquefaction occurred at depth (Jones, 2003). Liquefaction induced lateral spreading may occur at this site. According to Youd (2005), if the liquefiable layer lies at a depth greater that about twice the height of a free face, lateral spread is not likely to develop. Free faces occur along the All American Canal and West Side Main Canal embankments at this site. Lateral spreading is considered likely at this project site. Sand boils and lateral spreading were noted along the All American Canal and West Side Main Canal after the April 4, 2010 earthquake.

Mitigation: Ground improvement methods are available to mitigate liquefaction such as deep soil mixing (cement), vibro-compaction, vibro-replacement, geopiers, stone columns, compaction grouting, or deep dynamic compaction. Some other means to mitigate liquefaction damage include either a deep foundation system, rigid mat foundations and grade-beam reinforced foundations that can withstand some differential movement or tilting, but may not protect fracturing of buried utilities.

Because of the potential for differential settlement upon liquefaction, the designer should consider the structures be either founded on:

1) Foundations that use grade-beam footings to tie floor slabs and isolated columns to continuous footings (conventional or post-tensioned).

2) Structural flat-plate mats, either conventionally reinforced or tied with post-tensioned tendons.

These alternatives reduce the potential effects of liquefaction-induced settlements by making the structures more able to withstand differential settlement.

Liquefaction mitigation measures apply to structures potentially sensitive to differential settlement due to liquefaction such as maintenance/storage buildings. Liquefaction mitigation measures are not required for structures such as PV module piles and distributed inverter stations because the differential settlement of those structures is small and not expected to result in loss of integrity or functionality.

Section 4
RECOMMENDATIONS

4.1 Site Preparation

Clearing and Grubbing: All surface improvements, debris or vegetation including grass, trees, and weeds on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic strippings should be stockpiled and not used as engineered fill. All trash, construction debris, concrete slabs, old pavement, landfill, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign material by the grading contractor and removed under the supervision of the Geotechnical Engineer. Any excavations resulting from site clearing should be sloped to a bowl shape to the lowest depth of disturbance and backfilled under the observation of the geotechnical engineer's representative.

Building Pad Preparation: The exposed surface soil within the building pad/foundation areas should be removed to 30 inches below the building pad elevation or existing natural surface grade (whichever is lower) extending five feet beyond all exterior wall/column lines (including concreted areas adjacent to the building). Exposed subgrade should be scarified to a depth of 8 inches, uniformly moisture conditioned to 5 to 10% above optimum moisture content (clays) or ±2% above optimum (sands), and recompacted to 85 to 90% (clays) or a minimum of 90% (sands) of the maximum density determined in accordance with ASTM D1557 methods. Prior to over-excavation of the surface soil, deep moisture penetration may be achieved by bordering the site and applying multiple floodings or by sprinkler application to allow water to permeate to a minimum depth of 3.5 feet (20% minimum moisture content) below existing natural surface. Extended drying periods may be required when utilizing this method of pre-saturation.

The native soil is suitable for use as engineered fill provided it is free from concentrations of organic matter or other deleterious material. The fill soil should be uniformly moisture conditioned by discing and watering to the limits specified above, placed in maximum 8-inch lifts (loose), and compacted to the limits specified above. Clay soil should not be overcompacted because highly compacted soil will result in increased swelling. Imported fill soil (for foundations designed for expansive soil conditions) should have a Plasticity Index less than 25 and sulfates (SO_4) less than 2,000 ppm.

If foundation designs are to be utilized which do not include provisions for expansive soil, an engineered building support pad consisting of 3.0 feet of granular soil, placed in maximum 8-inch lifts (loose), compacted to a minimum of 90% of ASTM D1557 maximum density at 2% below to 4% above optimum moisture, should be placed below the bottom of the slab.

For foundations which do not include provisions for expansive soil conditions, non-expansive, granular soil meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35% passing the No. 200 sieve shall be used. The geotechnical engineer should approve imported fill soil sources before hauling material to the site. Imported granular fill should be placed in lifts no greater than 8 inches in loose thickness and compacted to a minimum of 90% of ASTM D1557 maximum dry density at optimum moisture ±2%.

In areas other than the building pad which are to receive area concrete slabs, the ground surface should be presaturated to a minimum depth of 24 inches and then scarified to 8 inches, moisture conditioned to a minimum of 5% over optimum, and recompacted to 83-87% of ASTM D1557 maximum density just prior to concrete placement.

On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill above pipezone, but may be difficult to uniformly maintain at specified moistures and compact to the specified densities. Native backfill should only be placed and compacted after encapsulating buried pipes with suitable bedding and pipe envelope material.

Imported granular material is acceptable for backfill of utility trenches. Granular trench backfill used in building pad areas should be plugged with a solid (no clods or voids) 2-foot width of native clay soils at each end of the building foundation to prevent landscape water migration into the trench below the building.

Backfill soil of utility trenches within paved areas should be placed in layers not more that 6 inches in thickness and mechanically compacted to a minimum of 87% of the ASTM D1557 maximum dry density except for the top 12 inches of the trench which shall be compacted to at least 90%.

Observation and Density Testing: All site preparation and fill placement should be continuously observed and tested by a representative of a qualified geotechnical engineering firm. Full-time observation services during the excavation and scarification process is necessary to detect undesirable materials or conditions and soft areas that may be encountered in the construction area. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "*geotechnical engineer of record*" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the recommendations for site development.

4.2 Foundations and Settlements

Shallow spread footings are suitable to support the new office/ maintenance building provided they are structurally tied with grade-beams to continuous perimeter wall footings to resist differential movement associated with expansive soils. Footings shall be founded on undisturbed native soil. The foundations may be designed using an allowable soil bearing pressure of 1,500 psf for compacted native clay soil. The allowable soil pressure may be increased by 20% for each foot of embedment depth in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 3,000 psf. Foundations should be designed for a maximum deflection of L/480.

As an alternative to shallow spread foundations, flat plate structural mats or grade-beam reinforced foundations may be used to mitigate expansive soil heave and/or liquefaction related movement.

Flat Plate Structural Mats: Flat plate structural mats may be used to mitigate expansive soils at the project site. The structural mat shall have a double mat of steel (minimum No. 4's @ 12" O.C. each way – top and bottom) and a minimum thickness of 10 inches. Mat edges shall have a minimum edge footing of 12 inches width and 18 inches depth (below the building pad surface). Mats may be designed by CBC Chapter 18, Section 1805.8.2 methods using an Effective Plasticity Index of 34.

Structural mats may be designed for a modulus of subgrade reaction (Ks) of 100 pci when placed on compacted clay or a subgrade modulus of 300 pci when placed on 3.0 feet of granular fill. Mats shall overlay 2 inches of sand and a 10-mil polyethylene vapor retarder. The building support pad shall be moisture conditioned and recompacted as specified in Section 4.1 of this report.

Grade-beam Reinforced Foundations: Specific soil data for structures with grade-beam reinforced foundations placed on the native clays (without replacement of the surface clays with 3.0 feet of granular fill or lime treated soil placed over native clays) are presented below in accordance with the design method given in CBC Chapter 18 (2007) Section 1805.8.2 (WRI/CRSI Design of Slab-on-Ground Foundations):

- Weighted Plasticity Index (PI) = 42
- Slope Coefficient (C_s) = 1.0
- Strength Coefficient (C_o) = 0.8
- Climatic Rating (C_w) = 15
- Effective PI = 34
- 1-C Value = 0.21
- Maximum Grade-beam Spacing = 16 feet

Post-tensioned Slabs: If post-tensioned slabs are considered for this project, the following soil criteria shall be used in the Post Tensioning Institute (PTI, 2004) design methods:

Maximum Edge Moisture Variation Distance, e_m	Center: 6.9 ft. Edge: 3.6 ft.
Differential Swell, y_m	Center: 0.23 in. Edge: 3.59 in.
Bearing Capacity:	1,500 psf
Maximum Allowable Slab Deflection	1 inch

Clamping devices and end anchors for post-tensioned tendons are susceptible to corrosion from aggressive soil and landscape water conditions. Therefore, a minimum concrete cover of 3.0 inches, a PVC end cap and epoxy coatings should be specified for the tendon ends with a positive bonding agent used with polymer modified cementitious material to patch the recessed anchor cup.

A complete encapsulation system intended for corrosive environments is the recommended protection method for post-tensioning cables and anchoring/clamping devices.

All exterior footings should be embedded a minimum of 18 inches below the building support pad or lowest adjacent final grade, whichever is deeper. Embedment depth of interior footings should be a minimum of 12 inches deep. Interior footing embedment depths shall be determined by the structural engineer/designer and should be sufficient to limit differential movement to 1.0 inch or less. Continuous wall footings should have a minimum width of 12 inches. Spread footings should have a minimum dimension of 24 inches and should be structurally tied to perimeter footings or grade beams. Recommended concrete reinforcement and sizing for all footings should be provided by the structural engineer.

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 250 pcf (300 pcf for imported sands) to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.25 (0.35 for imported sands) may also be used at the base of the footings to resist lateral loading.

Foundation movement under the estimated static (non-seismic) loadings and static site conditions are estimated to not exceed 1 inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed. Seismically induced liquefaction settlement of the surrounding land mass and structure may be on the order of 1 to 4½ inches.

4.3 Drilled Piers

Individual short piers should be adequate to support the solar panels. Embedment depth for short piers to resist lateral loads where no-constraint is provided at ground surface may be designed using the following formula per 2007 CBC Section 1805.7.2.1:

$$d = A/2\ [1 + (1+4.36h/A)^{1/2}]$$

where:

$A = 2.34P/S_1 b$
b = Pier diameter in feet
d = Embedment depth in feet (but not over 12 feet for purpose of computing lateral pressure)
h = Distance in feet from ground surface to point of application of "P"
P = Applied lateral force in pounds
S_1 = Allowable lateral soil bearing pressure (basic value of 100 psf/f (see 2007 CBC Table 1804.2). Isolated piers such solar panel short piers that are not adversely affected by a 0.5 inch motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral soil bearing pressures equal to two times the provided value.

The short pier foundations may be designed using an allowable soil bearing pressure of 1,500 psf for the native soils.

4.4 Driven Steel Piles

The use of driven steel piles requires special provisions for corrosion protection due to the corrosive nature of the subsurface soils. Precast, prestressed concrete piles are often used in the corrosive soil environments of the Imperial Valley. Selection of pile type may be based on drivability and cost comparisons.

The specified tip elevation (5 and 10 feet) and design load for a 6-inch driven steel circular pile are given in Table 4.

TABLE 4
Allowable Capacities of Pile Foundations

Pile Type:	Driven Circular Steel Pile (Diameter = 6")	
Specified Tip Depth (ft):	**5 feet**	**10 feet**
Pile Diameter:	6"	6"
Allowable Axial Capacity (tons) – FS=2.5:	2.9	5.3
Allowable Uplift Capacity (tons) – FS=2.5:	3.0	6.0
Allowable Lateral Capacity (tons) for ¼ inch deflection:		
Free Head Condition (kips):	3.7	5.5
Fixed Head Condition (kips):	9.2	11.1
Maximum Moments from Lateral Load,		
Free Head Condition (ft-kips):	4.2	8.9
Fixed Head Condition (ft-kips):	-21.7	-23.0
Depth of Maximum Moment,		
Free Head (ft):	2.2	3.1
Fixed Head (ft):	0	0

Recommendations for other pile types and sizes can be made available upon request.

Lateral Capacity: The allowable lateral capacity is based on a deflection of one-quarter inch at the top of the pile. If greater deflection can be tolerated, lateral load capacity can be increased directly in proportion to a maximum of one inch deflection.

Settlement: Total settlements of less than ¼ inch, and differential movement of about two-thirds of total movement for single piles designed according to the preceding recommendations. If pile spacing is at least 2.5 pile diameters center-to-center, no reduction in axial load capacity is considered necessary for a group effect.

4.5 Slabs-On-Grade

Concrete slabs and flatwork placed over native clay soil should be designed in accordance with Chapter 18 of the 2007 CBC and shall be a minimum of 5 inches thick due to expansive soil conditions (minimum 6-inch thick where the slab is subjected to wheel loads). Concrete floor slabs shall be monolithically placed with the footings (no cold joints). The concrete slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. The vapor retarder should be properly lapped and continuously sealed and extend a minimum of 12 inches into the footing excavations. The vapor retarder should be placed between 2 inches (above and below) of clean sand (Sand Equivalent SE>30). Concrete slabs may be placed without a sand cover directly over a 15-mil vapor retarder (Stego-Wrap or equivalent).

Concrete slab and flatwork reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 4 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking.

Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings. All steel components of the foundation system should be protected from corrosion by maintaining a 3-inch minimum concrete cover of densely consolidated concrete at footings (by use of a vibrator). The construction joint between the foundation and any mowstrips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint. Epoxy coated embedded steel components or permanent waterproofing membranes placed at the exterior footing sidewall may also be used to mitigate the corrosion potential of concrete placed in contact with native soil.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement. Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

All independent flatwork (sidewalks, patios) should be placed on a minimum of 2 inches of concrete sand or aggregate base, dowelled to the perimeter foundations where adjacent to the building and sloped 2% or more away from the building. A minimum of 24 inches of moisture conditioned (5% minimum above optimum) and 8 inches of compacted subgrade (83 to 87%) should underlie all independent flatwork. Flatwork which contains steel reinforcing (except wire mesh) should be underlain by a 10-mil (minimum) polyethylene separation sheet and at least a 2-inch sand cover. All flatwork should be jointed in square patterns and at irregularities in shape at a maximum spacing of 10 feet or the least width of the sidewalk.

4.6 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plates C-7 and C-8). The native soils were found to have low to severe levels of sulfate ion concentration (83 to 9,162 ppm). Sulfate ions in high concentrations can attack the cementitious material in concrete, causing weakening of the cement matrix and eventual deterioration by raveling. The California Building Code recommends that increased quantities of Type II Portland Cement be used at a low water/cement ratio when concrete is subjected to moderate sulfate concentrations. Type V Portland Cement and/or Type II/V cement with 25% flyash replacement is recommended when the concrete is subjected to soil with severe sulfate concentration.

A minimum of 6.0 sacks per cubic yard of concrete (4,500 psi) of Type V Portland Cement with a maximum water/cement ratio of 0.45 (by weight) should be used for concrete placed in contact with native soil on this project (sitework including streets, sidewalks, driveways, patios, and foundations). Admixtures may be required to allow placement of this low water/cement ratio concrete.

The native soil has low to very severe levels of chloride ion concentration (10 to 8,140 ppm). Chloride ions can cause corrosion of reinforcing steel, anchor bolts and other buried metallic conduits. Resistivity determinations on the soil indicate very severe potential for metal loss because of electrochemical corrosion processes. Mitigation of the corrosion of steel can be achieved by using steel pipes coated with epoxy corrosion inhibitors, asphaltic and epoxy coatings, cathodic protection or by encapsulating the portion of the pipe lying above groundwater with a minimum of 3 inches of densely consolidated concrete. ***No metallic water pipes or conduits should be placed below foundations.***

Foundation designs shall provide a minimum concrete cover of three (3) inches around steel reinforcing or embedded components (anchor bolts, etc.) exposed to native soil or landscape water (to 18 inches above grade). If the 3-inch concrete edge distance cannot be achieved, all embedded steel components (anchor bolts, etc.) shall be epoxy dipped for corrosion protection or a corrosion inhibitor and a permanent waterproofing membrane shall be placed along the exterior face of the exterior footings. ***Hold-down straps should not be used at foundation edges due to corrosion of metal at its protrusion from the slab edge.*** Additionally, the concrete should be thoroughly vibrated at footings during placement to decrease the permeability of the concrete.
All copper piping within 18 inches of ground surface shall be wrapped with two layers of 10 mil plumbers tape or sleeved with PVC piping to prevent contact with soil. The trap primer pipe shall be completely encapsulated in a PVC sleeve and Type K copper should be utilized if polyethylene tubing cannot be used. Fire protection piping (risers) should be placed outside of the building foundation.

4.7 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the Laguna Salada, Superstition Hills, and Imperial Faults. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.4 of this report.

4.8 Pavements

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should decide the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements. The site is dominated by surficial sands in the northwestern portion of the site and clay soils in the southeastern portion of the site. Pavement structural sections have been provided for each soil type.

Based on the current State of California CALTRANS method, an estimated R-value of 40 for the sandy soils and 5 for the clay soils and assumed traffic indices, the following tables provides our estimates for asphaltic concrete (AC) and Portland Cement Concrete (PCC) pavement sections.

All weather access roads should consist of a minimum of 6 inches of Caltrans Class 2 aggregate base placed over 12 inches of moisture conditioned (minimum 4% above optimum if clays) native clay soil compacted to a minimum of 90% (95% if sand subgrade) of the maximum dry density determined by ASTM D1557.

RECOMMENDED PAVEMENTS SECTIONS (CLAY SOILS)

R-Value of Subgrade Soil - 5 (estimated) Design Method - CALTRANS 2006

Traffic Index (assumed)	Flexible Pavements		(*) Flexible Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Asphaltic Concrete Thickness (in.)	Aggregate Base/Lime Thickness (in.)
4.0	3.0	6.5	3.0	4.0/14.0
5.0	3.0	9.0	3.0	4.0/15.0
6.0	3.0	14.0	3.0	6.0/18.0
6.5	4.0	14.0	4.0	6.0/18.0
8.0	4.0	18.0	4.0	8.0/21.0
10.0	4.5	26.0	4.5	13.0/24.0
11.0	5.5	28.0	5.5	15.0/24.0

(*) Pavement structural section when used with 12 inches of lime-treated subgrade soil (3-6% quicklime by weight) compacted to 95% minimum with minimum Unconfined Compressive Strength of 55 psi.

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum (½ inch maximum for parking areas), medium grading with PG64-16 asphalt cement, compacted to a minimum of 95% of the Hveem density (CAL 366).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (minimum 4% above optimum if clays) native clay soil compacted to a minimum of 90% (95% if sand subgrade) of the maximum dry density determined by ASTM D1557. No additional subgrade preparation is required for soil-lime mixtures.
4) Typical Street Classifications (Imperial County)

Parking Areas:	TI = 4.0
Cul-de-Sacs:	TI = 5.0
Local Streets:	TI = 6.0
Minor Collectors:	TI = 6.5
Major Collectors:	TI = 8.0
Minor Arterial:	TI = 10.0
Primary Arterial:	TI = 11.0

PAVEMENT STRUCTURAL SECTIONS (SAND SOILS)

R-Value of Subgrade Soil - 40 Design Method - CALTRANS 2006

Traffic Index (assumed)	Flexible Pavements		Rigid (PCC) Pavements	
	Asphaltic Concrete Thickness (in.)	Aggregate Base Thickness (in.)	Concrete Thickness (in.)	Aggregate Base Thickness (in.)
4.0	3.0	4.0	5.0	4.0
5.0	3.0	4.0	5.0	4.0
6.0	3.0	6.0	5.5	4.0
7.0	3.5	8.0	6.0	6.0
8.0	3.5	10.0	7.0	6.0
9.0	4.0	12.0	7.5	6.0
10.0	4.5	14.0	8.0	6.0

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum (½ inch maximum for parking areas), medium grading with PG64-16 asphalt cement, compacted to a minimum of 95% of the Hveem density (CAL 366).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (minimum of optimum moisture) native sandy silt soil compacted to a minimum of 95% of the maximum dry density determined by ASTM D1557.
4) Portland cement concrete for pavements should have Type V cement, a minimum compressive strength of 4,000 psi at 28 days, and a maximum water-cement ratio of 0.50.
5) Typical Street Classifications (Imperial County)

Parking Areas:	TI = 4.0
Cul-de-Sacs:	TI = 5.0
Local Streets:	TI = 6.0
Minor Collectors:	TI = 6.5
Major Collectors:	TI = 8.0
Minor Arterial:	TI = 10.0
Primary Arterial:	TI = 11.0

Section 5
LIMITATIONS

5.1 Limitations

The recommendations and conclusions within this report are based on current information regarding the proposed Imperial Solar Energy Center South project located south of State Route 98 around the intersection of Pulliam Road and Anza Road approximately 7 miles west of Calexico, California. The conclusions and recommendations of this report are invalid if:

- Structural loads change from those stated or the structures are relocated.
- The Additional Services section of this report is not followed.
- This report is used for adjacent or other property.
- Changes of grade or groundwater occur between the issuance of this report and construction other than those anticipated in this report.
- Any other change that materially alters the project from that proposed at the time this report was prepared.

Findings and recommendations in this report are based on selected points of field exploration, geologic literature, laboratory testing, and our understanding of the proposed project. Our analysis of data and recommendations presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. If detected, these conditions may require additional studies, consultation, and possible design revisions.

This report contains information that may be useful in the preparation of contract specifications. However, the report is not worded is such a manner that we recommend its use as a construction specification document without proper modification. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in Imperial County at the time the report was prepared. No express or implied warranties are made in connection with our services.

This report should be considered invalid for periods after two years from the report date without a review of the validity of the findings and recommendations by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice.

The client has responsibility to see that all parties to the project including, designer, contractor, and subcontractor are made aware of this entire report. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

5.2 Additional Services

We recommend that a qualified geotechnical consultant be retained to provide the tests and observations services during construction. *The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.*

The professional opinions presented in this report are based on the assumption that:

- Consultation during development of design and construction documents to check that the geotechnical professional opinions are appropriate for the proposed project and that the geotechnical professional opinions are properly interpreted and incorporated into the documents.
- Landmark Consultants will have the opportunity to review and comment on the plans and specifications for the project prior to the issuance of such for bidding.
- Observation, inspection, and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, building pad and subgrade preparation, and backfilling of utility trenches.
- Observation of foundation excavations and reinforcing steel before concrete placement.
- Other consultation as necessary during design and construction.

We emphasize our review of the project plans and specifications to check for compatibility with our professional opinions and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

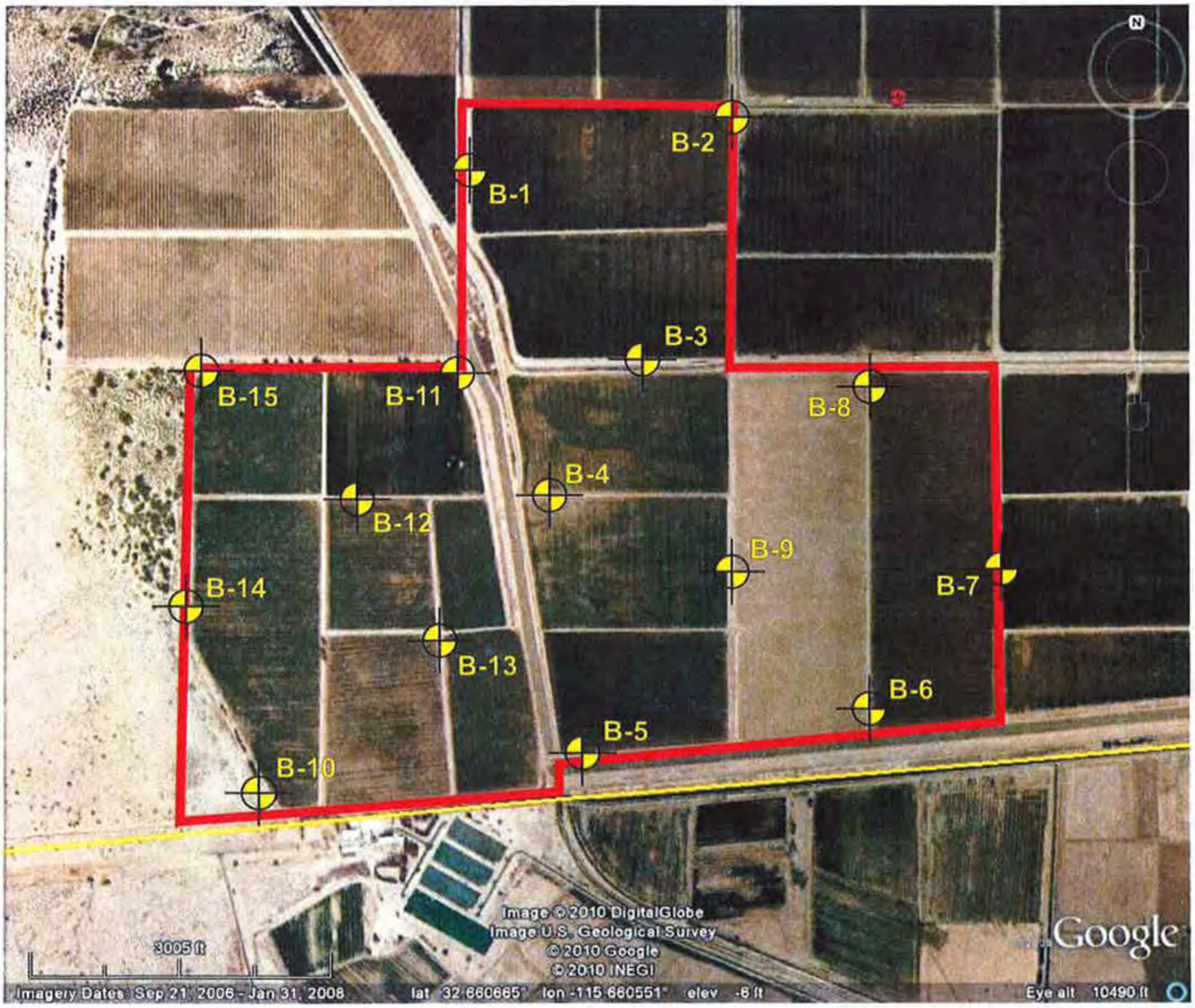
APPENDIX A

LANDMARK
Geo-Engineers and Geologists

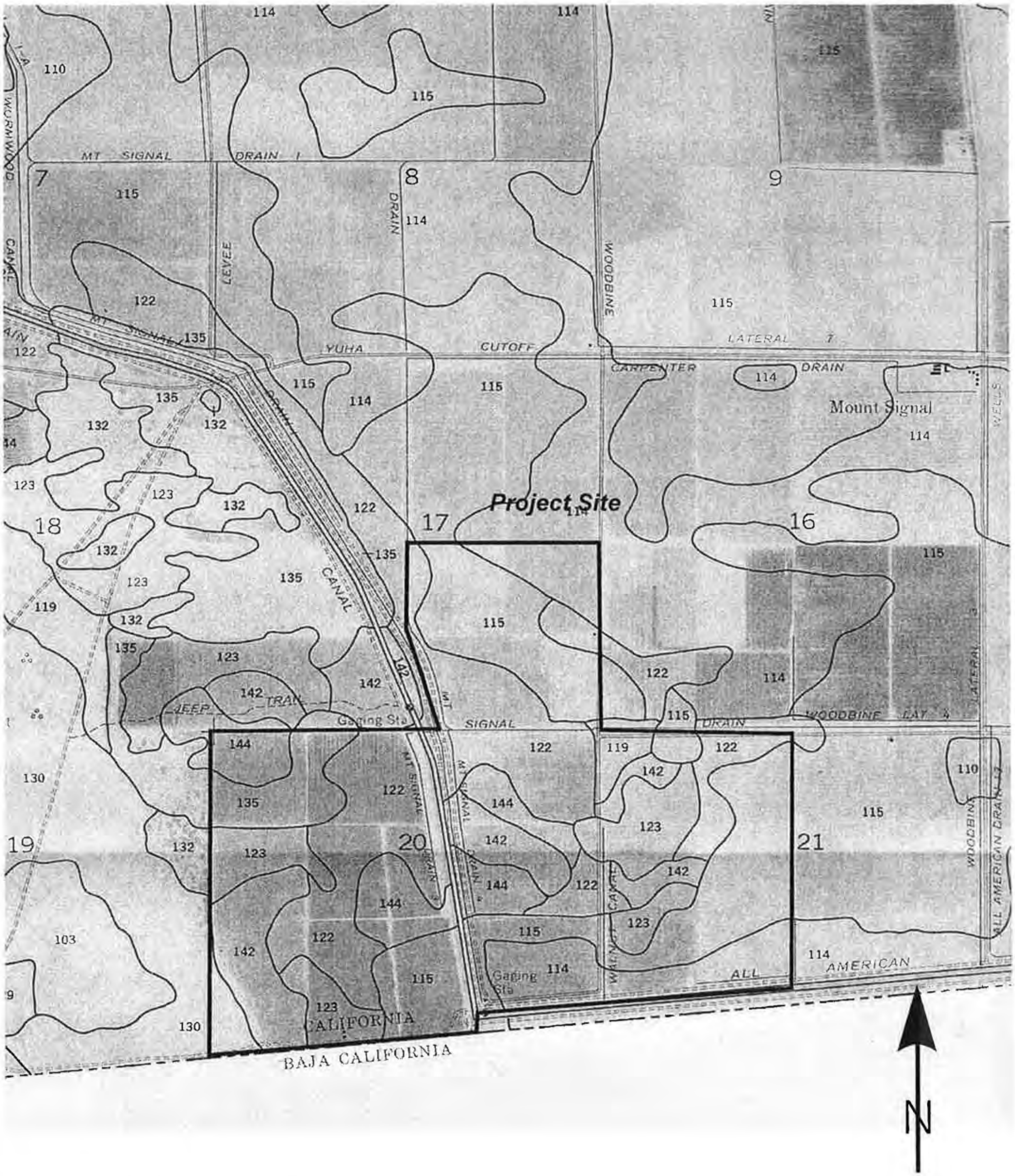
Project No.: LE10094

Vicinity Map

Plate A-1



LANDMARK Geo-Engineers and Geologists **Project No.: LE10094**	**Boring Location Map**	**Plate A-2**



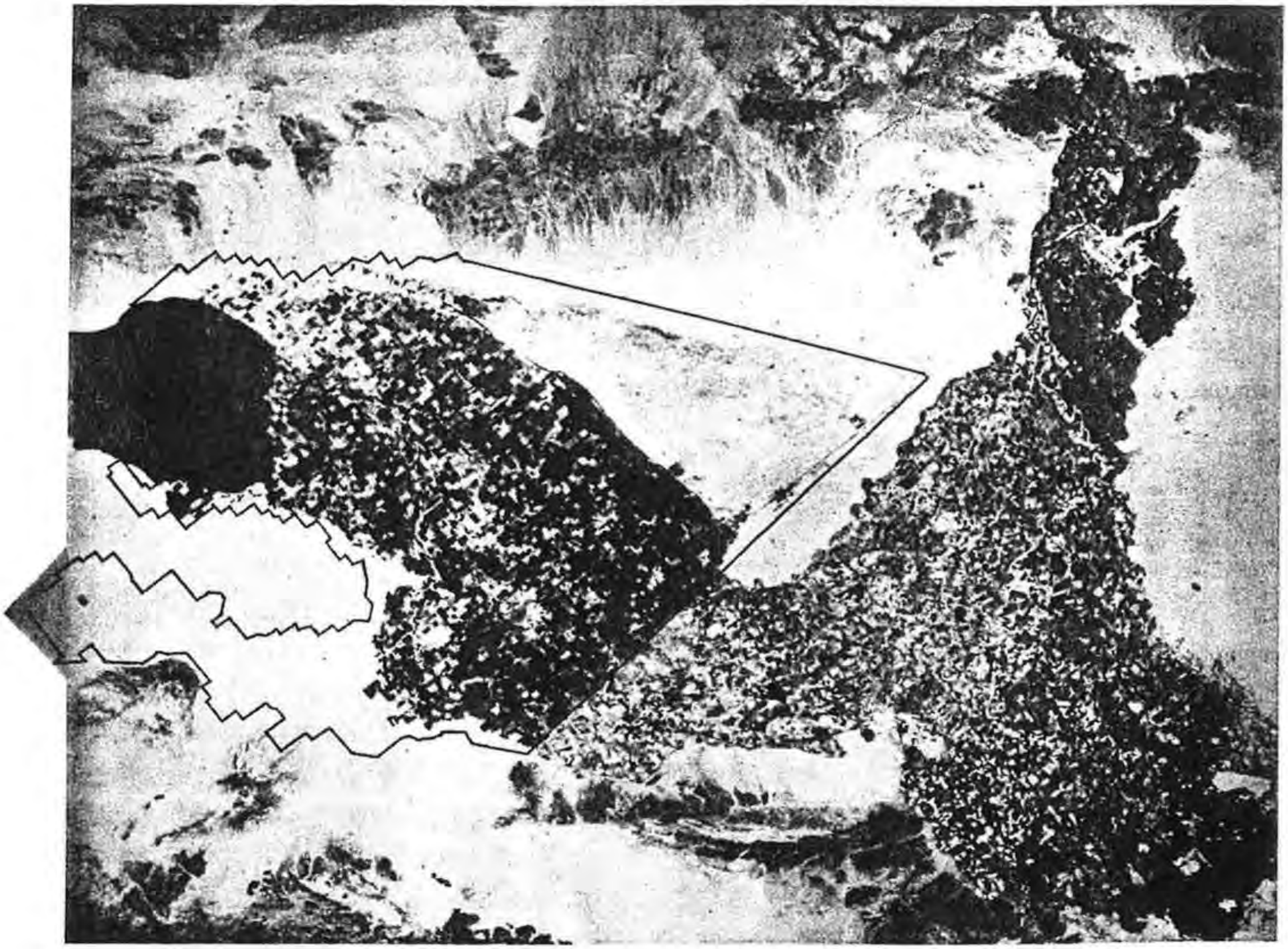
LANDMARK
Geo-Engineers and Geologists
Project No.: LE10094

Soil Survey Map

Plate A-3

Soil Survey of

IMPERIAL COUNTY CALIFORNIA IMPERIAL VALLEY AREA



United States Department of Agriculture Soil Conservation Service
in cooperation with
University of California Agricultural Experiment Station
and
Imperial Irrigation District

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

Soil name and map symbol	Depth	USDA texture	Classification		Fragments > 3 inches	Percentage passing sieve number--				Liquid limit	Plasticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
111*:											
Holtville----------	0-10	Silty clay loam	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	10-22	Clay, silty clay	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
	22-60	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	65-85	25-35	NP-10
Imperial-----------	0-12	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
112---------------- Imperial	0-12	Silty clay-------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
113---------------- Imperial	0-12	Silty clay-------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay, clay, silty clay loam.	CH	A-7	0	100	100	100	85-95	50-70	25-45
114---------------- Imperial	0-12	Silty clay-------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
115*:											
Imperial-----------	0-12	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
Glenbar------------	0-13	Silty clay loam	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
116*:											
Imperial-----------	0-13	Silty clay loam	CL	A-7	0	100	100	100	85-95	40-50	10-20
	13-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
Glenbar------------	0-13	Silty clay loam	CL	A-6, A-7	0	100	100	90-100	70-95	35-45	15-25
	13-60	Clay loam, silty clay loam.	CL	A-6	0	100	100	90-100	70-95	35-45	15-30
117, 118----------- Indio	0-12	Loam-------------	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-72	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
119*:											
Indio--------------	0-12	Loam-------------	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-72	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
Vint---------------	0-10	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	25-35	---	NP
	10-60	Loamy sand, loamy fine sand.	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
120*--------------- Laveen	0-12	Loam-------------	ML, CL-ML	A-4	0	100	95-100	75-85	55-65	20-30	NP-10
	12-60	Loam, very fine sandy loam.	ML, CL-ML	A-4	0	95-100	85-95	70-80	55-65	15-25	NP-10

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

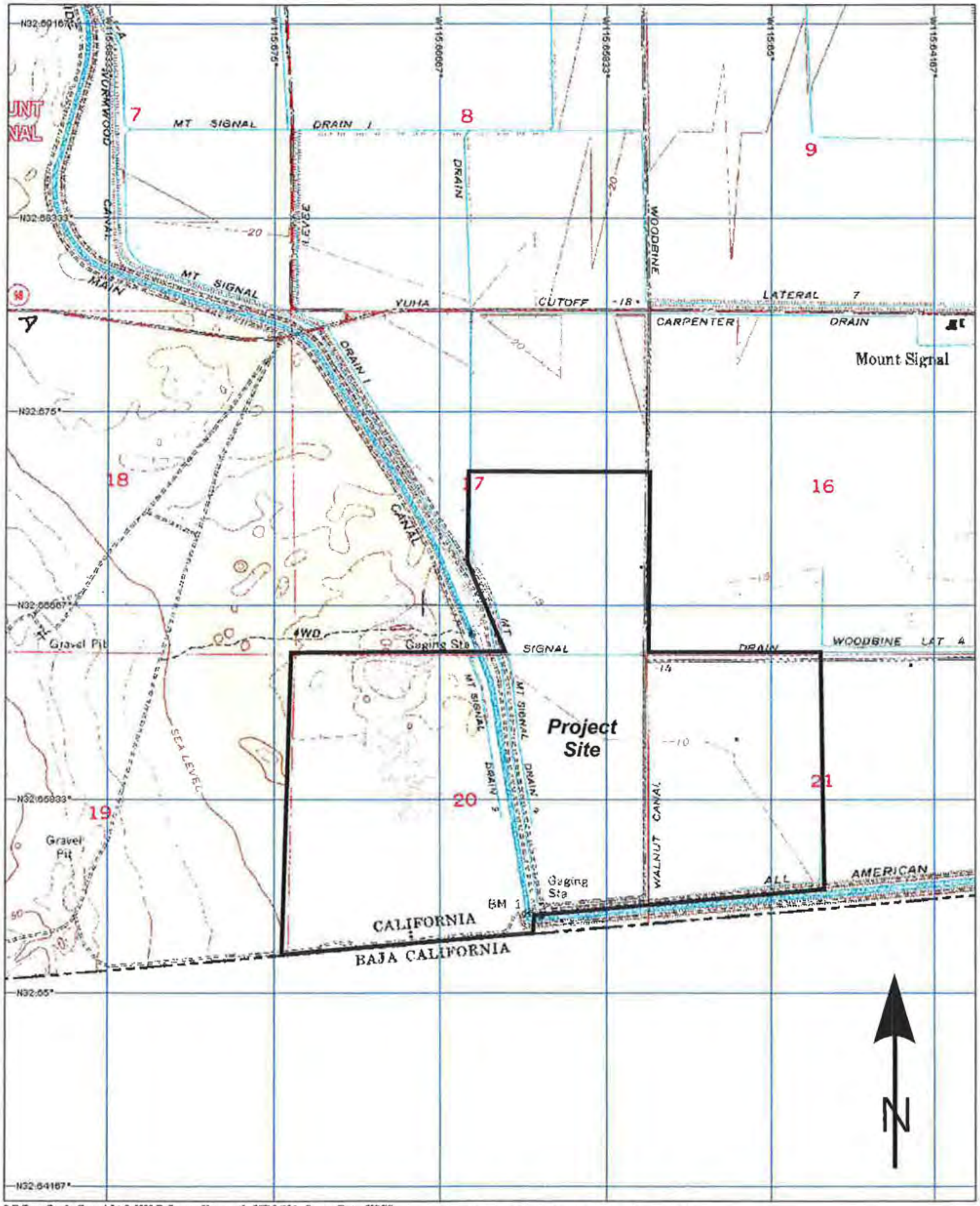
Soil name and map symbol	Depth	USDA texture	Classification		Frag-ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas-ticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
121----------------- Meloland	0-12	Fine sand--------	SM, SP-SM	A-2, A-3	0	95-100	90-100	75-100	5-30	---	NP
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-65	25-35	NP-10
	26-71	Clay, silty clay, silty clay loam.	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-40
122----------------- Meloland	0-12	Very fine sandy loam.	ML	A-4	0	95-100	95-100	95-100	55-85	25-35	NP-10
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-70	25-35	NP-10
	26-71	Clay, silty clay, silty clay loam.	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-40
123*:											
Meloland-----------	0-12	Loam-------------	ML	A-4	0	95-100	95-100	95-100	55-85	25-35	NP-10
	12-26	Stratified loamy fine sand to silt loam.	ML	A-4	0	100	100	90-100	50-70	25-35	NP-10
	26-38	Clay, silty clay, silty clay loam.	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-40
	38-60	Stratified silt loam to loamy fine sand.	SM, ML	A-4	0	100	100	75-100	35-55	25-35	NP-10
Holtville----------	0-12	Loam-------------	ML	A-4	0	100	100	85-100	55-95	25-35	NP-10
	12-24	Clay, silty clay	CH, CL	A-7	0	100	100	95-100	85-95	40-65	20-35
	24-36	Silt loam, very fine sandy loam.	ML	A-4	0	100	100	95-100	55-85	25-35	NP-10
	36-60	Loamy very fine sand, loamy fine sand.	SM, ML	A-2, A-4	0	100	100	75-100	20-55	---	NP
124, 125----------- Niland	0-23	Gravelly sand---	SM, SP-SM	A-2, A-3	0	90-100	70-95	50-65	5-25	---	NP
	23-60	Silty clay, clay, clay loam.	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
126----------------- Niland	0-23	Fine sand--------	SM, SP-SM	A-2, A-3	0	90-100	90-100	50-65	5-25	---	NP
	23-60	Silty clay-------	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
127----------------- Niland	0-23	Loamy fine sand	SM	A-2	0	90-100	90-100	50-65	15-30	---	NP
	23-60	Silty clay-------	CL, CH	A-7	0	100	100	85-100	80-95	40-65	20-40
128*:											
Niland-------------	0-23	Gravelly sand---	SM, SP-SM	A-2, A-3	0	90-100	70-95	50-65	5-25	---	NP
	23-60	Silty clay, clay, clay loam.	CL, CH	A-7	0	100	100	85-100	80-100	40-65	20-40
Imperial-----------	0-12	Silty clay-------	CH	A-7	0	100	100	100	85-95	50-70	25-45
	12-60	Silty clay loam, silty clay, clay.	CH	A-7	0	100	100	100	85-95	50-70	25-45
129*: Pits											
130, 131----------- Rositas	0-27	Sand-------------	SP-SM	A-3, A-1, A-2	0	100	80-100	40-70	5-15	---	NP
	27-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP

See footnote at end of table.

TABLE 11.--ENGINEERING INDEX PROPERTIES--Continued

Soil name and map symbol	Depth	USDA texture	Classification		Frag- ments > 3 inches	Percentage passing sieve number--				Liquid limit	Plas- ticity index
			Unified	AASHTO		4	10	40	200		
	In				Pct					Pct	
132, 133, 134, 135-Rositas	0-9	Fine sand-------	SM	A-3, A-2	0	100	80-100	50-80	10-25	---	NP
	9-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
136----------------Rositas	0-4	Loamy fine sand	SM	A-1, A-2	0	100	80-100	40-85	10-35	---	NP
	4-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
137----------------Rositas	0-12	Silt loam-------	ML	A-4	0	100	100	90-100	70-90	20-30	NP-5
	12-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
138*:											
Rositas------------	0-4	Loamy fine sand	SM	A-1, A-2	0	100	80-100	40-85	10-35	---	NP
	4-60	Sand, fine sand, loamy sand.	SM, SP-SM	A-3, A-2, A-1	0	100	80-100	40-85	5-30	---	NP
Superstition------	0-6	Loamy fine sand	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
139----------------Superstition	0-6	Loamy fine sand	SM	A-2	0	100	95-100	70-85	15-25	---	NP
	6-60	Loamy fine sand, fine sand, sand.	SM	A-2	0	100	95-100	70-85	15-25	---	NP
140*: Torriorthents											
Rock outcrop											
141*: Torriorthents											
Orthids											
142----------------Vint	0-10	Loamy very fine sand.	SM, ML	A-4	0	100	100	85-95	40-65	15-25	NP-5
	10-60	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
143----------------Vint	0-12	Fine sandy loam	ML, CL-ML, SM, SM-SC	A-4	0	100	100	75-85	45-55	15-25	NP-5
	12-60	Loamy sand, loamy fine sand.	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
144*:											
Vint--------------	0-10	Very fine sandy loam.	SM, ML	A-4	0	100	100	85-95	40-65	15-25	NP-5
	10-40	Loamy fine sand	SM	A-2	0	95-100	95-100	70-80	20-30	---	NP
	40-60	Silty clay------	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35
Indio-------------	0-12	Very fine sandy loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	12-40	Stratified loamy very fine sand to silt loam.	ML	A-4	0	95-100	95-100	85-100	75-90	20-30	NP-5
	40-72	Silty clay------	CL, CH	A-7	0	100	100	95-100	85-95	40-65	20-35

* See description of the map unit for composition and behavior characteristics of the map unit.



3-D TopoQuads Copyright © 1999 DeLorme Yarmouth, ME 04096 Source Data: USGS

700 ft Scale: 1 : 24,000 Detail: 13-1 Datum: WGS84

LANDMARK Geo-Engineers and Geologists **Project No.: LE10094**	**Topographic Map**	**Plate A-4**

APPENDIX B

LOG OF BORING No. 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY SAND (SM): Brown, dry to moist, fine grained sand.			
5			9	4.5	SILTY CLAY (CL): Brown, moist, hard, 1" sand layer at 6 ft.			
10			26	4.5	Anticipated GW=13.0 ft	100.5	24.8	c = 2.11 tsf
15			39		SILTY SAND (SM): Lt. brown, saturated, dense. No recovery.			
20			12	4.5	SILTY CLAY (CL): Brown, moist, hard, thin sand interbeds (1-2").			
25					Total Depth = 21.5' Groundwater was encountered at 15.0 ft at the time of exploration but may raise with time to about 13.0 ft bgs. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 04/29/10 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: +/- 13.0 ft.

LOGGED BY: S. Williams TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.

SURFACE ELEVATION: -30 ft HAMMER WT.: 140 lbs. DROP: 30 in.

LOG OF BORING No. 2

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5		CL	20	2.5	SILTY CLAY (CL): Brown, moist, very stiff, medium plasticity Anticipated GW=8.0 ft	97.1	27.4	c = 1.01 tsf
10			19	2.5		100.9	24.1	LL=42% PI=27%
15			11	3.0				
20		ML	3		CLAYEY SILT (ML): Brown, saturated, very loose.			
25			4		some fine grained sand.			SAND=22% FINES=78%
30		SM	13		SILTY SAND (SM): Lt. brown, saturated, medium dense, fine to medium grained.	16.8		LL=NV PI=NP
35		CL	13	2.5	SILTY CLAY (CL): Reddish brown, very moist, very stiff, medium plasticity.			
40			10	2.0				
45		SM/CH	9		SILTY SAND/CLAY (SM/CH): Brown, saturated, loose/stiff, thinly interbedded 2-4" thick.			
50		CH	8	4.5	CLAY (CH): Brown, moist, hard, high plasticity.			
55					Total Depth = 51.5' Groundwater was encountered at 16.5 ft at the time of exploration but may raise with time to about 8.0 ft bgs. Backfilled with excavated soil			
60								

DATE DRILLED:	04/29/10	TOTAL DEPTH:	51.5 Feet	DEPTH TO WATER:	+/- 8.0 ft.
LOGGED BY:	S. Williams	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-14 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

LANDMARK Geo-Engineers and Geologists

PLATE B-2

LOG OF BORING No. 3

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY SAND (SM): Tan, dry to damp, fine grained sand.			
5			14		SILTY SAND/SILTY CLAY (SMCL): Tan/brown, moist, medium dense/stiff, fine to medium grained sand, interbedded.			
10			13	0.5	SILTY CLAY (CL): Brown, moist, hard, thin sand interbeds (1-2"). Anticipated GW=12.5 ft	99.4	24.3	LL=27% PI=14%
15			3	2.5				
20			10	2.5				
25					Total Depth = 21.5' Groundwater was encountered at 12.5 ft at the time of exploration. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 04/29/10 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: +/- 12.5 ft.

LOGGED BY: S. Williams TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.

SURFACE ELEVATION: -11 ft HAMMER WT.: 140 lbs. DROP: 30 in.

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	LOG OF BORING No. 4 SHEET 1 OF 1 DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5		SM	25		SILTY SAND (SM): Brown, very moist to saturated, medium dense, fine grained sand. Anticipated GW=5 ft			
10		CL	18	4.0	SILTY CLAY (CL): Brown, very moist, hard.	96.0	28.6	c = 0.76 tsf
15		SM	10		SILTY SAND (SM): Grayish brown, saturated, medium dense, fine grained sand, fossiliferous, 1" clay layer at 16.4 ft.			SAND=88% FINES=12%
20		CL	10	2.5	SILTY CLAY (CL): Reddish brown, very moist, very stiff.			
25					Total Depth = 21.5' Groundwater was encountered at 10.5 ft at the time of exploration, but saturated at 6 ft. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 04/29/10 | TOTAL DEPTH: 21.5 Feet | DEPTH TO WATER: +/- 5 ft.

LOGGED BY: S. Williams | TYPE OF BIT: Hollow Stem Auger | DIAMETER: 8 in.

SURFACE ELEVATION: -5 ft | HAMMER WT.: 140 lbs. | DROP: 30 in.

PROJECT No. LE10094

LANDMARK Geo-Engineers and Geologists

PLATE B-4

LOG OF BORING No. 5

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY CLAY (CL): Brown, moist to very moist, stiff to very stiff.			
					Anticipated GW=5 ft			
5			14	1.5		101.9	23.6	c = 0.74 tsf
10			14	2.0		103.4	24.6	LL=42 PI=27%
15			6	2.5				
					SILTY SAND (SM): Brown, saturated.			
20			3	1.5	SILTY CLAY (CL): Reddish brown, very moist, stiff.			
25					Total Depth = 21.5' Groundwater was encountered at 18.5 ft at the time of exploration. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 04/29/10 TOTAL DEPTH: 21.5 Feet DEPTH TO WATER: +/- 5 ft.

LOGGED BY: S. Williams TYPE OF BIT: Hollow Stem Auger DIAMETER: 8 in.

SURFACE ELEVATION: +4 ft HAMMER WT.: 140 lbs. DROP: 30 in.

PROJECT No. LE10094

LANDMARK
Geo-Engineers and Geologists

PLATE B-5

LOG OF BORING No. 6

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					CLAY (CH): Brown, moist to very moist, stiff to very stiff.			
5			18	2.0	Anticipated GW=8 ft			
10			10	1.0		101.2	23.3	LL=65% PI=44%
15			15	1.0				
20			6	1.0	CLAYEY SILT/SILTY CLAY (ML/CL): Brown, moist, loose/stiff.			
25					Total Depth = 21.5' Groundwater was not encountered at the time of exploration, but may rise to about 8 ft. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 04/29/10 | TOTAL DEPTH: 21.5 Feet | DEPTH TO WATER: +/- 8 ft.

LOGGED BY: J. Avalos | TYPE OF BIT: Hollow Stem Auger | DIAMETER: 8 in.

SURFACE ELEVATION: -3 ft | HAMMER WT.: 140 lbs. | DROP: 30 in.

PROJECT No. LE10094

LANDMARK Geo-Engineers and Geologists

PLATE B-6

LOG OF BORING No. 7

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY CLAY (CL): Brown, moist, very stiff, medium plasticity			
5			11		CLAYEY SILT (ML): Brown, very moist, medium dense.	22.7	104.6	
				1.0	Anticipated GW=8.0 ft			
10			20	2.0	SILTY CLAY (CL): Brown, moist, very stiff, medium plasticity	30.4	91.3	c = 0.76 tsf
15			8	2.0				
20			9		SANDY SILT (ML): Brown, saturated, loose, fine grained sand.			
25			5		clayey			
30			7			28.0		LL=26% PI=2%
35			11	2.0	CLAY (CH): Reddish brown, very moist, very stiff, high plasticity.			
40			8	3.0				
45			8	2.5	some silty clay			
50			14	3.5				
55					Total Depth = 51.5' Groundwater was encountered at 18.5 ft at the time of exploration but may raise with time to about 8.0 ft bgs. Backfilled with excavated soil			
60								

DATE DRILLED:	04/29/10	TOTAL DEPTH:	51.5 Feet	DEPTH TO WATER:	+/- 8.0 ft.
LOGGED BY:	J. Avalos	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-8 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

LANDMARK Geo-Engineers and Geologists

PLATE B-7

LOG OF BORING No. 8

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
		CL			SILTY CLAY (CL): Brown, moist to very moist, stiff.			
5			26	1.5		109.6	14.9	c = 0.23 tsf
					Anticipated GW=8 ft			
10			8	1.5		100.5	24.3	LL=37% PI=23%
15			6	2.0				
		SM			SILTY SAND (SM): Brown, saturated, loose, fine grained.			
20		CL	6	2.0	SILTY CLAY (CL): Reddish brown, very moist, stiff.			
25					Total Depth = 21.5' Groundwater was encountered at 15.5 ft at the time of exploration. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED:	04/29/10	TOTAL DEPTH:	21.5 Feet	DEPTH TO WATER:	+/- 8 ft.
LOGGED BY:	S. Williams	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-11 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

LANDMARK Geo-Engineers and Geologists

PLATE B-8

LOG OF BORING No. 9

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
0–4		CL			SILTY CLAY (CL): Brown, moist, stiff, medium plasticity.			
5		SM	12		SILTY SAND (SM): Lt. brown, saturated, medium dense, fine grained. Anticipated GW=5 ft			
10		CL		3.0	SILTY CLAY (CL): Reddish brown, very moist, very stiff.			
		SM	29		SILTY SAND (SM): Brown, saturated, medium dense, fine grained.			
15		CH	7	3.0	CLAY (CH): Dark brown, very moist, very stiff.			LL=60% PI=42%
20		CH	6	2.5				

Total Depth = 21.5'
Groundwater was encountered at 15.0 ft at the time of exploration.
Backfilled with excavated soil

DATE DRILLED:	04/30/10	TOTAL DEPTH:	21.5 Feet	DEPTH TO WATER:	+/- 5 ft.
LOGGED BY:	S. Williams	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-5 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

LANDMARK
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PLATE B-9

LOG OF BORING No. 10

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
					SILTY CLAY (CL): Brown, dry, very stiff, medium plasticity			
					SILTY SAND (SM): Dark brown, moist, fine grained sand.			
5			16					
					SANDY SILT (ML): Brown, moist, medium dense, fine grained sand.			
10			30	3.0	SILTY CLAY (CL): Brown, moist, very stiff to hard, medium plasticity	106.9	20.7	c = 1.43 tsf
15			21	4.0	Anticipated GW=16.0 ft	106.1	22.3	
20				2.0				
			17		SILT (ML): Brown, very moist, medium dense.			
25			9	3.0	CLAY (CH): Reddish brown, very moist, very stiff to hard, high plasticity.			
30			11	4.0				
35			20	4.0				
40			72		SILTY SAND (SM): Dark brown, moist, very dense, fine grained sand.			
45			10		medium dense			
50					No recovery			
55					Total Depth = 51.5' Groundwater was encountered at 32 ft at the time of exploration but may raise with time to about 16.0 ft bgs. Backfilled with excavated soil			
60								

DATE DRILLED:	04/30/10	TOTAL DEPTH:	51.5 Feet	DEPTH TO WATER:	+/- 8.0 ft.
LOGGED BY:	J. Avalos	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	+2 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

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PLATE B-10

LOG OF BORING No. 11

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
5			20	4.0	SILTY CLAY (CL): Brown, very moist, hard to stiff, medium plasticity Anticipated GW=8.0 ft	23.2	100.3	c = 0.75 tsf
10			9	2.0		26.6	95.3	
15			10	1.5				
20			4		CLAYEY SILT/SILT (ML): Brown, saturated, loose.			
25			8			28.5		LL=NV% PI=NP%
30			13					
35			5		SILTY SAND (SM): Dark brown, moist, loose, fine grained sand.			
				3.0	CLAY (CH): Reddish brown, very moist, very stiff to hard, high plasticity.			
40			6	4.0				
45			10		CLAYEY SILT/SILT (ML): Brown, saturated, loose.	22.0		LL=25% PI=1%
50			9	2.5	CLAY (CH): Reddish brown, very moist, very stiff, high plasticity.			
55			9		Total Depth = 51.5' Groundwater was encountered at 20.5 ft at the time of exploration but may raise with time to about 8.0 ft bgs. Backfilled with excavated soil			
60								

DATE DRILLED:	04/30/10	TOTAL DEPTH:	51.5 Feet	DEPTH TO WATER:	+/- 8.0 ft.
LOGGED BY:	J. Avalos	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-7 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

LANDMARK
Geo-Engineers and Geologists

PLATE B-11

LOG OF BORING No. 12

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
0–3		CL			SILTY CLAY (CL): Brown, moist to very moist, stiff to very stiff.			
5		SM	25		Anticipated GW=9 ft SILTY SAND (SM): Lt. brown, moist to saturated, medium dense to dense, fine grained.			
10			16					SAND=86% FINES=14%
15			35			109.1	20.0	
20		CH	10	3.0	CLAY (CH): Reddish brown, very moist, very stiff.			
25					Total Depth = 21.5' Groundwater was encountered at 13 feet at the time of exploration, but may rise to about 9 ft. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED:	04/30/10	TOTAL DEPTH:	21.5 Feet	DEPTH TO WATER:	+/- 9 ft.
LOGGED BY:	J. Avalos	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-5 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

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PLATE B-12

LOG OF BORING No. 13

SHEET 1 OF 1

DEPTH	FIELD: SAMPLE	FIELD: USCS CLASS.	FIELD: BLOW COUNT	FIELD: POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	LABORATORY: DRY DENSITY (pcf)	LABORATORY: MOISTURE CONTENT (% dry wt.)	LABORATORY: OTHER TESTS
		CL			SILTY CLAY (CL): Brown, moist to very moist, stiff to very stiff.			
5			18	1.5		101.2	20.5	c = 0.69 tsf
10				3.0				
		SM	24		Anticipated GW=13 ft			
15			25		SILTY SAND (SM): Lt. brown, moist to saturated, medium dense, fine grained.			
20		CH	7	2.5	CLAY (CH): Reddish brown, very moist, very stiff.			
25					Total Depth = 21.5' Groundwater was encountered at 18 feet at the time of exploration, but may rise to about 13 ft. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED: 04/30/10 — TOTAL DEPTH: 21.5 Feet — DEPTH TO WATER: +/- 13 ft.

LOGGED BY: J. Avalos — TYPE OF BIT: Hollow Stem Auger — DIAMETER: 8 in.

SURFACE ELEVATION: -1 ft — HAMMER WT.: 140 lbs. — DROP: 30 in.

PROJECT No. LE10094

Geo-Engineers and Geologists

PLATE B-13

LOG OF BORING No. 14

SHEET 1 OF 1

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5		SM	47		SILTY SAND (SM): Lt. brown, moist, dense to medium dense, fine to coarse grained, some gravel.	106.7	3.4	SAND=77% FINES=23%
10			48		very fine grained sand.			
15			29		saturated Anticipated GW=16 ft			
20			38					
25					Total Depth = 21.5' Groundwater was encountered at 18 feet at the time of exploration, but may rise to about 16 ft. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED:	04/30/10	TOTAL DEPTH:	21.5 Feet	DEPTH TO WATER:	+/- 16 ft.
LOGGED BY:	J. Avalos	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-1 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

LANDMARK Geo-Engineers and Geologists

PLATE B-14

LOG OF BORING No. 15

DEPTH	SAMPLE	USCS CLASS.	BLOW COUNT	POCKET PEN. (tsf)	DESCRIPTION OF MATERIAL	DRY DENSITY (pcf)	MOISTURE CONTENT (% dry wt.)	OTHER TESTS
5		SM	24		SILTY SAND (SM): Lt. brown, moist, medium dense, fine grained.			
					Anticipated GW=9 ft			
10			28		saturated	111.0	18.2	SAND=89% FINES=11%
15			7					
		CH		3.5	CLAY (CH): Reddish brown, moist, very stiff.			
20		SM	12		SILTY SAND (SM): Brown, saturated, medium dense, fine grained.			
25					Total Depth = 21.5' Groundwater was encountered at 13 feet at the time of exploration, but may rise to about 9 ft. Backfilled with excavated soil			
30								
35								
40								
45								
50								
55								
60								

DATE DRILLED:	04/30/10	TOTAL DEPTH:	21.5 Feet	DEPTH TO WATER:	+/- 9 ft.
LOGGED BY:	J. Avalos	TYPE OF BIT:	Hollow Stem Auger	DIAMETER:	8 in.
SURFACE ELEVATION:	-7 ft	HAMMER WT.:	140 lbs.	DROP:	30 in.

PROJECT No. LE10094

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PLATE B-15

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
Coarse grained soils More than half of material is larger that No. 200 sieve	Gravels	Clean gravels (less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
	More than half of coarse fraction is larger than No. 4 sieve		GP	Poorly graded gravels, or gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	Sands	Clean sands (less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
	More than half of coarse fraction is smaller than No. 4 sieve		SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
Fine grained soils More than half of material is smaller than No. 200 sieve	Silts and clays		ML	Inorganic silts, clayey silts with slight plasticity
	Liquid limit is less than 50%		CL	Inorganic clays of low to medium plasticity, gravely, sandy, or lean clays
			OL	Organic silts and organic clays of low plasticity
	Silts and clays		MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts
	Liquid limit is more than 50%		CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			PT	Peat and other highly organic soils

GRAIN SIZES

Silts and Clays	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

200 40 10 4 3/4" 3" 12"

US Standard Series Sieve — Clear Square Openings

Sands, Gravels, etc.	Blows/ft. *
Very Loose	0-4
Loose	4-10
Medium Dense	10-30
Dense	30-50
Very Dense	Over 50

Clays & Plastic Silts	Strength **	Blows/ft. *
Very Soft	0-0.25	0-2
Soft	0.25-0.5	2-4
Firm	0.5-1.0	4-8
Stiff	1.0-2.0	8-16
Very Stiff	2.0-4.0	16-32
Hard	Over 4.0	Over 32

* Number of blows of 140 lb. hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 in. I.D.) split spoon (ASTM D1586).

** Unconfined compressive strength in tons/s.f. as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D1586), Pocket Penetrometer, Torvane, or visual observation.

Type of Samples:

Ring Sample — Standard Penetration Test — Shelby Tube — Bulk (Bag) Sample

Drilling Notes:

1. Sampling and Blow Counts
   Ring Sampler - Number of blows per foot of a 140 lb. hammer falling 30 inches.
   Standard Penetration Test - Number of blows per foot.
   Shelby Tube - Three (3) inch nominal diameter tube hydraulically pushed.
2. P. P. = Pocket Penetrometer (tons/s.f.).
3. NR = No recovery.
4. GWT = Ground Water Table observed @ specified time.

LANDMARK Geo-Engineers and Geologists **Project No. LE10094**	**Key to Logs**	**Plate B-16**

APPENDIX C

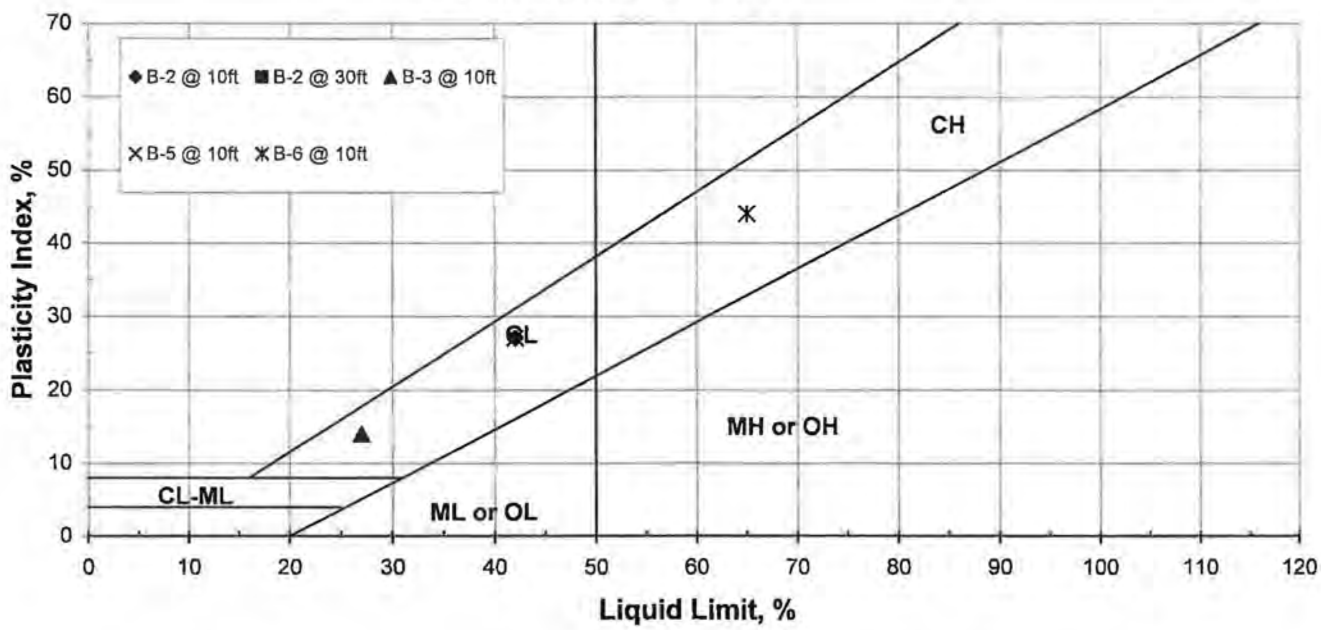
LANDMARK CONSULTANTS, INC.

CLIENT: CSOLAR Development, LLC
PROJECT: Imperial Valley South Solar Farm
JOB No.: LE10094
DATE: 05/24/10

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
B-2	10	42	15	27	CL
B-2	30	NV	NP		ML
B-3	10	27	13	14	CL
B-5	10	42	15	27	CL
B-6	10	65	21	44	CH

PLASTICITY CHART



LANDMARK
Geo-Engineers and Geologists

Project No.: LE10094

Atterberg Limits Test Results

Plate C-1

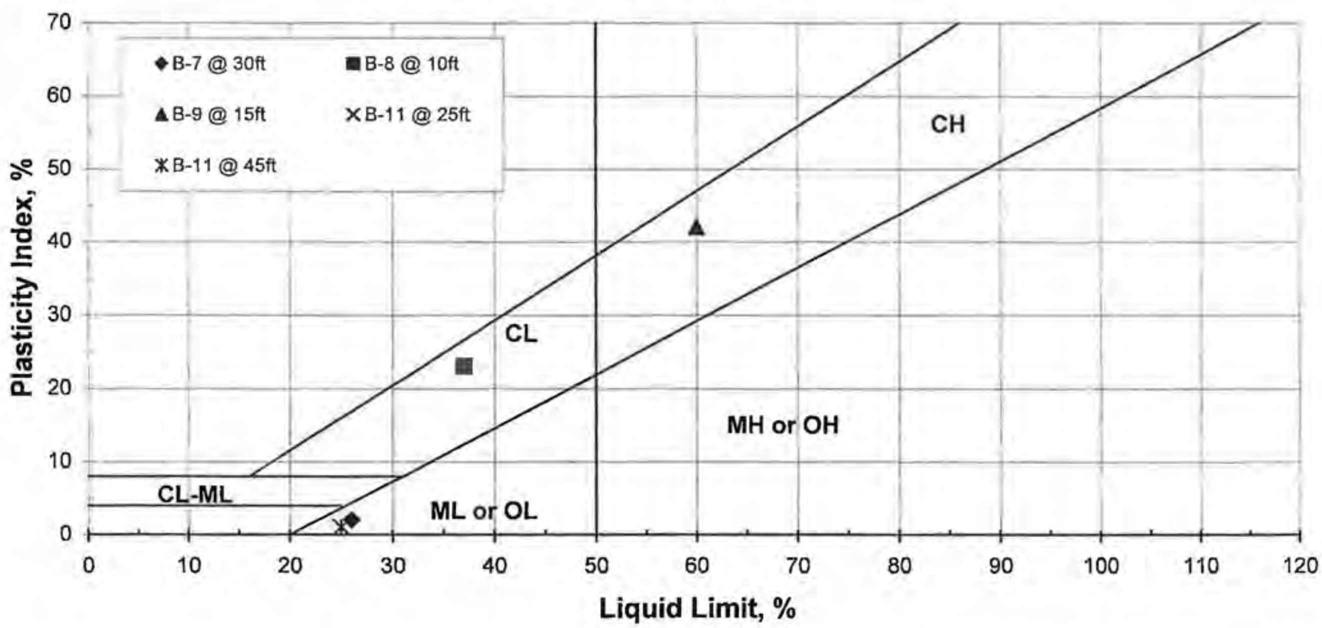
LANDMARK CONSULTANTS, INC.

CLIENT: CSOLAR Development, LLC
PROJECT: Imperial Valley South Solar Farm
JOB No.: LE10094
DATE: 05/24/10

ATTERBERG LIMITS (ASTM D4318)

Sample Location	Sample Depth (ft)	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	USCS Classification
B-7	30	26	24	2	ML
B-8	10	37	14	23	CL
B-9	15	60	18	42	CH
B-11	25	NV	NP		ML
B-11	45	25	24	1	ML

PLASTICITY CHART



LANDMARK
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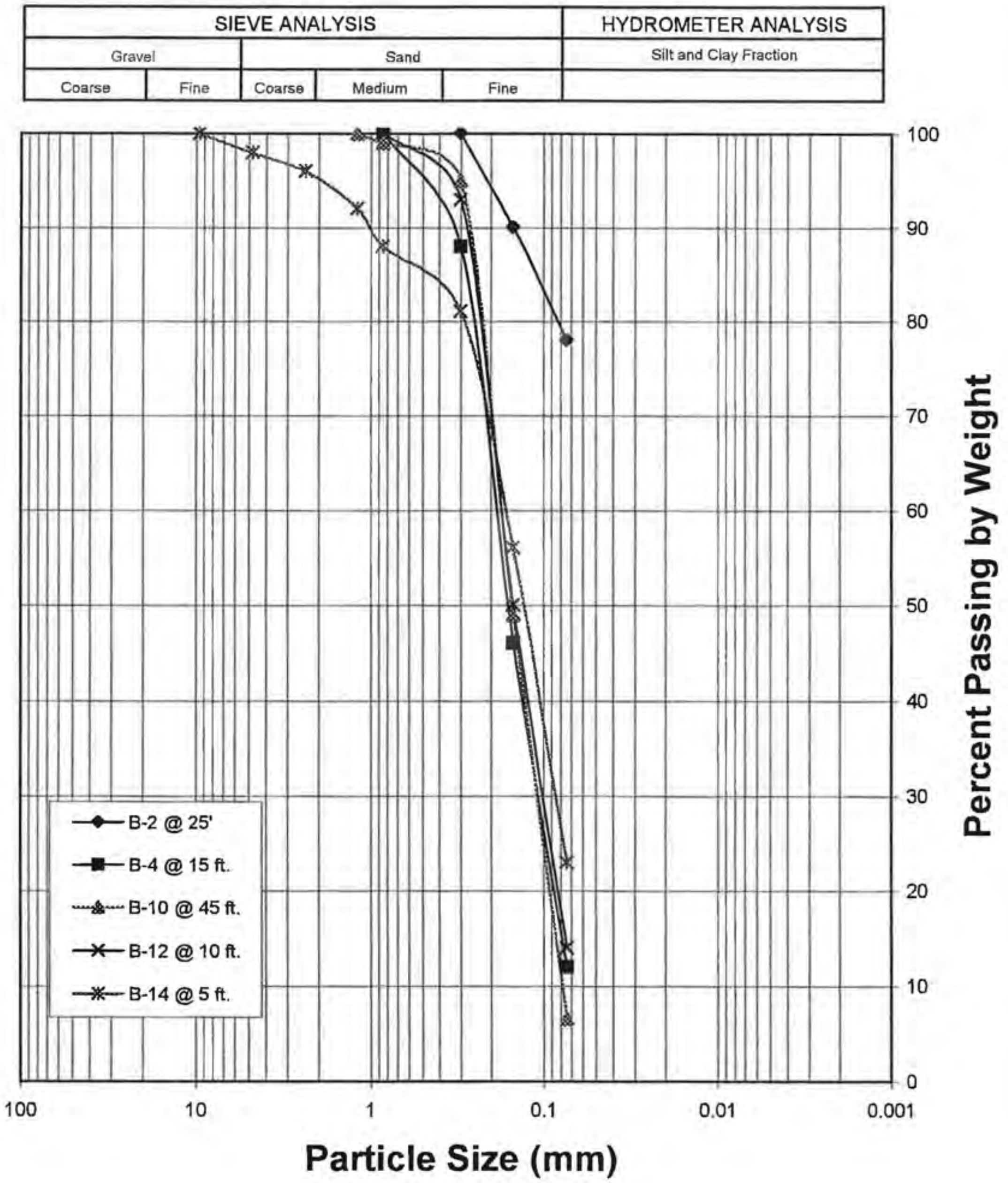
Project No.: LE10094

Atterberg Limits Test Results

Plate C-2



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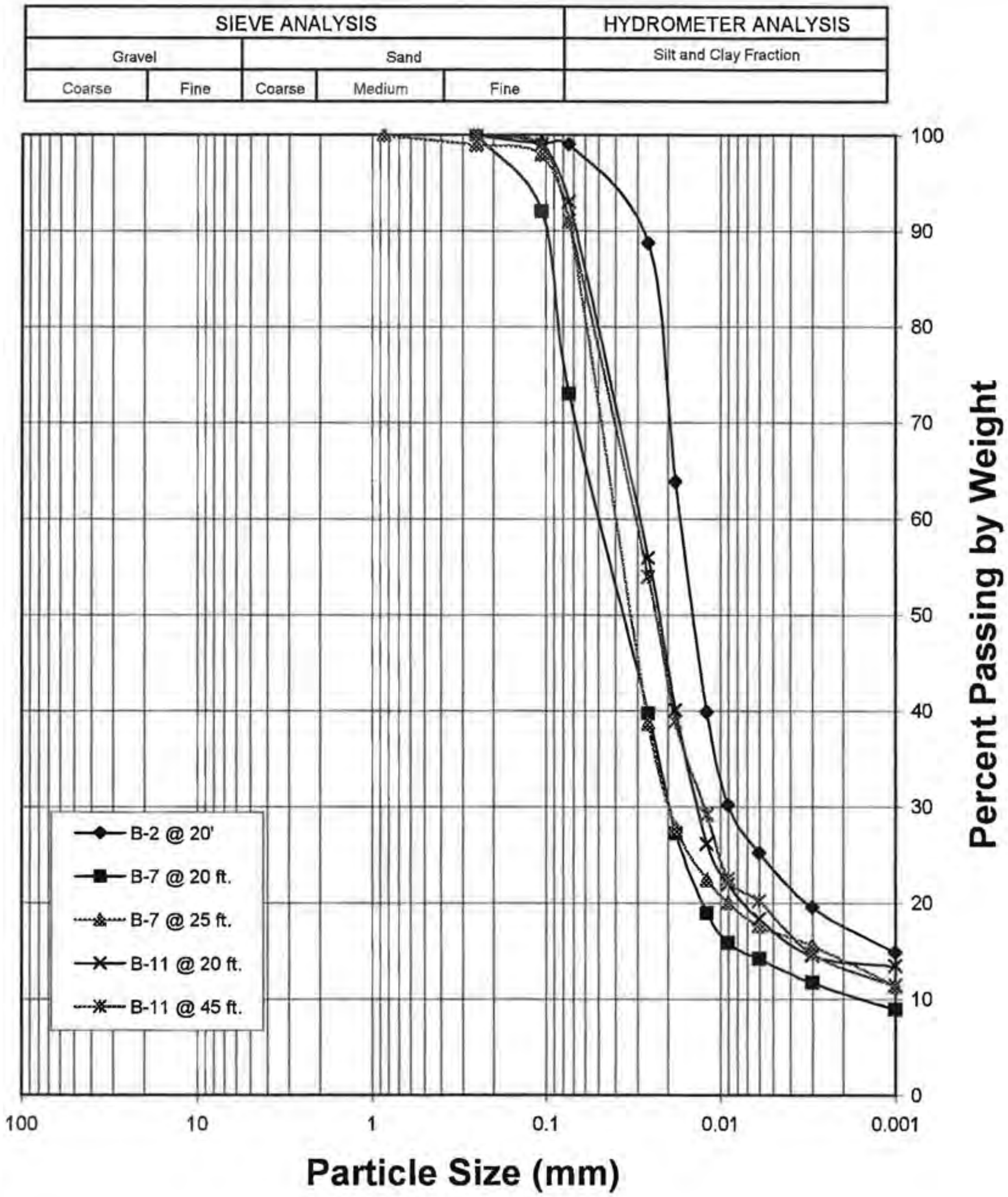
Project No.: LE10094

Grain Size Analysis

Plate C-3



LANDMARK Geo-Engineers and Geologists **Project No.: LE10094**	**Grain Size Analysis**	**Plate** **C-4**

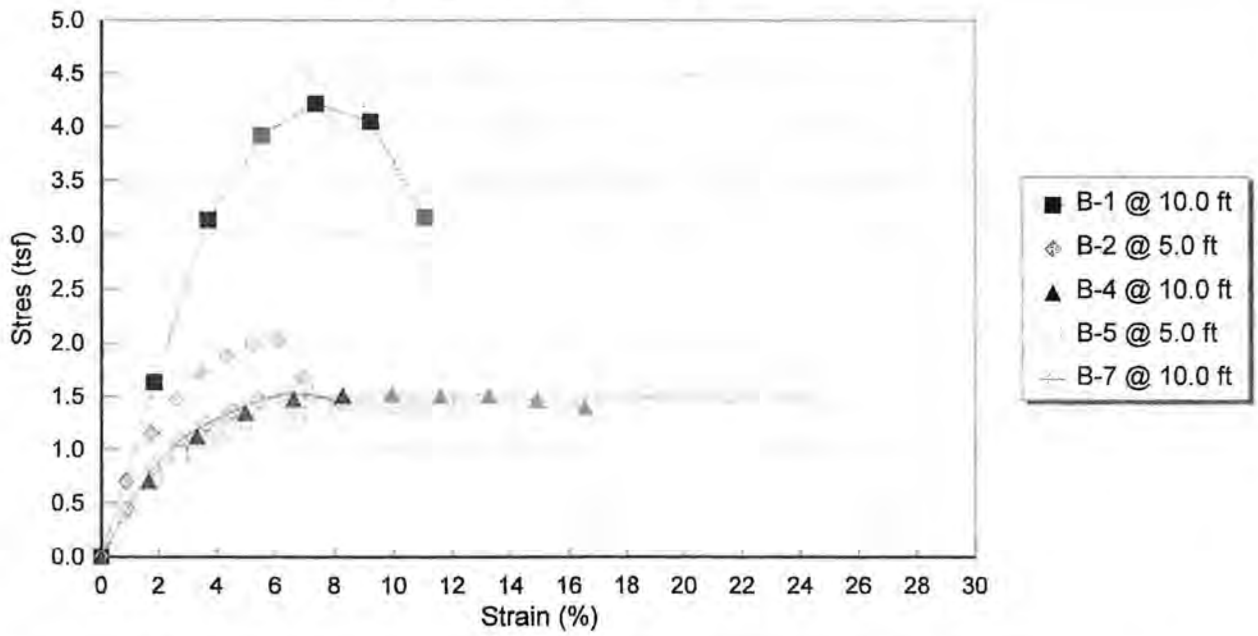
LANDMARK CONSULTANTS, INC.

CLIENT: CSOLAR Development, LLC
PROJECT: Imperial Valley South Solar Farm
JOB NO: LE10094
DATE: 05/21/10

UNCONFINED COMPRESSION TEST (ASTM D2166)

Boring No.	Sample Depth (ft)	Natural Moisture Content (%)	Unit Dry Weight (pcf)	Maximum Compressive Strength (tsf)	Cohesion (tsf)	Failure Strain (%)
B-1	10.0	24.8	100.5	4.22	2.11	7.4
B-2	5.0	27.4	97.1	2.03	1.01	6.1
B-4	10.0	28.6	96.0	1.52	0.76	9.9
B-5	5.0	23.6	101.9	1.48	0.74	16.3
B-7	10.0	30.4	91.3	1.53	0.76	6.3

STRESS-STRAIN PLOT



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Geo-Engineers and Geologists

Project No: LE10094

Unconfined Compression Test Results

Plate C-5

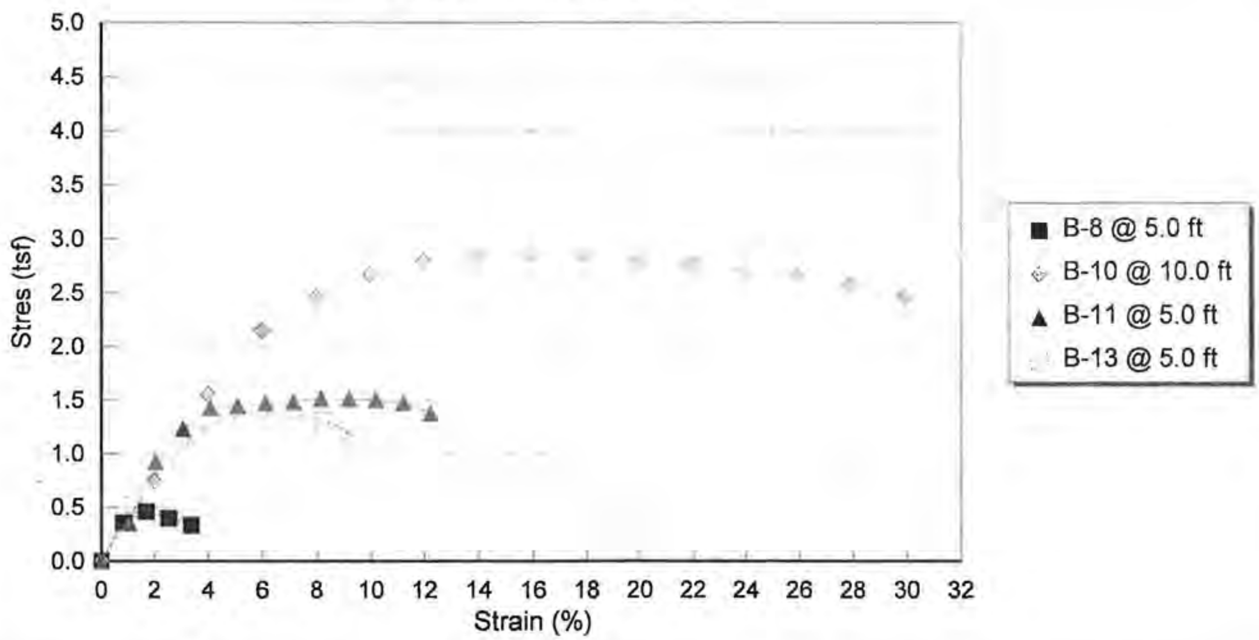
LANDMARK CONSULTANTS, INC.

CLIENT: CSOLAR Development, LLC
PROJECT: Imperial Valley South Solar Farm
JOB NO: LE10094
DATE: 05/21/10

UNCONFINED COMPRESSION TEST (ASTM D2166)

Boring No.	Sample Depth (ft)	Natural Moisture Content (%)	Unit Dry Weight (pcf)	Maximum Compressive Strength (tsf)	Cohesion (tsf)	Failure Strain (%)
B-8	5.0	14.9	109.6	0.46	0.23	1.7
B-10	10.0	20.7	106.9	2.86	1.43	15.9
B-11	5.0	23.2	100.3	1.51	0.75	8.1
B-13	5.0	20.5	101.2	1.37	0.69	5.9

STRESS-STRAIN PLOT



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Geo-Engineers and Geologists

Project No: LE10094

Unconfined Compression Test Results

Plate C-6

LANDMARK CONSULTANTS, INC.

CLIENT: CSOLAR Development, LLC
PROJECT: Imperial Valley South Solar Farm
JOB No.: LE10094
DATE: 05/24/10

CHEMICAL ANALYSIS

Boring:	B-2	B-4	B-6	B-8	**Caltrans**
Sample Depth, ft:	0-5	0-5	0-5	0-5	**Method**
pH:	7.1	8.1	7.4	7.5	**643**
Electrical Conductivity (mmhos):	5.74	0.45	2.36	1	**424**
Resistivity (ohm-cm):	100	1400	260	480	**643**
Chloride (Cl), ppm:	8,140	80	1,540	300	**422**
Sulfate (SO4), ppm:	9,162	83	2,148	763	**417**

General Guidelines for Soil Corrosivity

Material Affected	Chemical Agent	Amount in Soil (ppm)	Degree of Corrosivity
Concrete	Soluble Sulfates	0 - 1,000	Low
		1,000 - 2,000	Moderate
		2,000 - 20,000	Severe
		> 20,000	Very Severe
Normal Grade Steel	Soluble Chlorides	0 - 200	Low
		200 - 700	Moderate
		700 - 1,500	Severe
		> 1,500	Very Severe
Normal Grade Steel	Resistivity	1 - 1,000	Very Severe
		1,000 - 2,000	Severe
		2,000 - 10,000	Moderate
		> 10,000	Low

LANDMARK
Geo-Engineers and Geologists

Project No.: LE10094

Selected Chemical Test Results

Plate C-7

LANDMARK CONSULTANTS, INC.

CLIENT: CSOLAR Development, LLC
PROJECT: Imperial Valley South Solar Farm
JOB No.: LE10094
DATE: 05/24/10

CHEMICAL ANALYSIS

Boring:	B-10	B-11	B-13	B-15	**Caltrans**
Sample Depth, ft:	0-5	0-5	0-5	0-5	**Method**
pH:	7.7	8.4	7.4	7.6	**643**
Electrical Conductivity (mmhos):	0.88	0.18	1.74	0.93	**424**
Resistivity (ohm-cm):	580	3700	510	540	**643**
Chloride (Cl), ppm:	350	10	460	310	**422**
Sulfate (SO4), ppm:	630	127	2,907	790	**417**

General Guidelines for Soil Corrosivity

Material Affected	Chemical Agent	Amount in Soil (ppm)	Degree of Corrosivity
Concrete	Soluble Sulfates	0 - 1,000	Low
		1,000 - 2,000	Moderate
		2,000 - 20,000	Severe
		> 20,000	Very Severe
Normal Grade Steel	Soluble Chlorides	0 - 200	Low
		200 - 700	Moderate
		700 - 1,500	Severe
		> 1,500	Very Severe
Normal Grade Steel	Resistivity	1 - 1,000	Very Severe
		1,000 - 2,000	Severe
		2,000 - 10,000	Moderate
		> 10,000	Low

LANDMARK
Geo-Engineers and Geologists

Project No.: LE10094

Selected Chemical Test Results

Plate C-8

APPENDIX D

APPENDIX E

Liquefaction Evaluation and Settlement Calculation

Project Name: IV South Solar Site -- Imperial County, CA
Project No.: LE10094
Location: B-2

Maximum Credible Earthquake	7	
Design Ground Motion	0.37	g
Total Unit Weight,	115	pcf
Water Unit Weight,	62.4	pcf
Depth to Groundwater	8	ft
Hammer Effenciency	90	
Required Factor of Safety	1.0	

Boring Data						Sampling Corrections							Corrected	Fines	SPT Clean	Cyclical	Cyclical	Factor	Volumetric	Induced
Depth		Blow Counts		Liquefiable	Overburden	Sampler	SPT	Energy	Borehole	Rod	Liner	Overburden	SPT	Content	Sands	Resistance	Stress	of	Strain (%)	Subsidence
(ft)	(m)	SPT	Mod. Cal.	Soil (0 / 1)	Pressure	Diameter	N_m	C_E	C_B	C_R	C_L	C_N	$(N_1)_{60}$	%	$(N_1)_{60CS}$	$CRR_{M7.5}$	CSR	Safety		(inch)
5	1.52		20	0	575	0.67	13	1.5	1.0	0.75	1	1.70	26	90	36		0.238	Non-Liq.	0.00	0.00
10	3.05		19	0	1025	0.67	13	1.5	1.0	0.80	1	1.36	21	90	30	0.435	0.264	1.96	0.00	0.00
15	4.57	11		0	1288	1	11	1.5	1.0	0.85	1.1	1.11	17	90	26	0.291	0.312	1.11	0.00	0.00
20	6.10	3		1	1551	1	3	1.5	1.0	0.95	1.1	0.96	5	70	10	0.113	0.341	0.39	2.60	1.56
25	7.62	4		1	1814	1	4	1.5	1.0	0.95	1.1	0.86	5	78	11	0.124	0.359	0.41	2.40	1.44
30	9.14	13		1	2077	1	13	1.5	1.0	0.95	1.1	0.78	16	25	22	0.241	0.368	0.78	1.37	0.82
35	10.67	13		0	2340	1	13	1.5	1.0	1.00	1.1	0.73	16	90	24	0.262	0.368	0.85	0.00	0.00
40	12.19	10		0	2603	1	10	1.5	1.0	1.00	1.1	0.68	11	90	18	0.199	0.362	0.66	0.00	0.00
45	13.72	9		0	2866	1	9	1.5	1.0	1.00	1.1	0.64	9	80	16	0.177	0.349	0.61	0.00	0.00
50	15.24	8		0	3129	1	8	1.5	1.0	1.00	1.1	0.61	8	95	15	0.158	0.333	0.57	0.00	0.00
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils* , Technical Report NCEER-97-0022, December 31, 1997.

Total Settlement **3.82**

Corrections to SPT (Modified from Skempton, 1986) as listed by Robertson and Wride.

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma_{VO})^{0.5}$ $C_N<=2$
Energy Ratio	Donut Hammer	C_E	0.5 to 1.0
	Safety Hammer		0.7 to 1.2
	Automatic-trip Donut type Hammer		0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch	C_B	1
	6 inch		1.05
	8 inch		1.15
Rod Length	10 feet to 13 feet	C_R	0.75
	13 feet to 19.8 ft.		0.85
	19.8 ft. to 33 ft.		0.95
	33 ft. to 98 ft.		1
	> 98 ft.		<1.0
Sampling Method	Standard Sampler	C_L	1
	Sampler without liners		1.1 to 1.3

Liquefaction Evaluation and Settlement Calculation

Project Name: IV South Solar Site -- Imperial County, CA
Project No.: LE10094
Location: B-7

Maximum Credible Earthquake	7
Design Ground Motion	0.37 g
Total Unit Weight,	115 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	8 ft
Hammer Effenciency	90
Required Factor of Safety	1.0

Boring Data						Sampling Corrections							Corrected	Fines	SPT Clean	Cyclical	Cyclical	Factor	Volumetric	Induced
Depth		Blow Counts		Liquefiable	Overburden	Sampler	SPT	Energy	Borehole	Rod	Liner	Overburden	SPT	Content	Sands	Resistance	Stress	of	Strain (%)	Subsidence
(ft)	(m)	SPT	Mod. Cal.	Soil (0 / 1)	Pressure	Diameter	N_m	C_E	C_B	C_R	C_L	C_N	$(N_1)_{60}$	%	$(N_1)_{60CS}$	$CRR_{M7.5}$	CSR	Safety		(inch)
5	1.52		11	0	575	0.67	7	1.5	1.0	0.75	1	1.70	14	90	22	0.239	0.238	Non-Liq.	0.00	0.00
10	3.05		20	0	1025	0.67	13	1.5	1.0	0.80	1	1.36	22	90	31		0.264	Non-Liq.	0.00	0.00
15	4.57	8		0	1288	1	8	1.5	1.0	0.85	1.1	1.11	12	90	20	0.215	0.312	0.82	0.00	0.00
20	6.10	9		1	1551	1	9	1.5	1.0	0.95	1.1	0.96	14	70	21	0.231	0.341	0.81	1.43	0.86
25	7.62	5		1	1814	1	5	1.5	1.0	0.95	1.1	0.86	7	70	13	0.141	0.359	0.47	2.15	1.29
30	9.14	7		1	2077	1	7	1.5	1.0	0.95	1.1	0.78	9	70	15	0.166	0.368	0.54	1.91	1.15
35	10.67	11		0	2340	1	11	1.5	1.0	1.00	1.1	0.73	13	90	21	0.225	0.368	0.73	0.00	0.00
40	12.19	8		0	2603	1	8	1.5	1.0	1.00	1.1	0.68	9	90	16	0.170	0.362	0.56	0.00	0.00
45	13.72	8		0	2866	1	8	1.5	1.0	1.00	1.1	0.64	8	80	15	0.164	0.349	0.56	0.00	0.00
50	15.24	14		0	3129	1	14	1.5	1.0	1.00	1.1	0.61	14	95	22	0.238	0.333	0.85	0.00	0.00
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31, 1997.

Total Settlement **3.29**

Corrections to SPT (Modified from Skempton, 1986) as listed by Robertson and Wride.

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma_{VO})^{0.5}$ C_N<=2
Energy Ratio	Donut Hammer	C_E	0.5 to 1.0
	Safety Hammer		0.7 to 1.2
	Automatic-trip Donut type Hammer		0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch	C_B	1
	6 inch		1.05
	8 inch		1.15
Rod Length	10 feet to 13 feet	C_R	0.75
	13 feet to 19.8 ft.		0.85
	19.8 ft. to 33 ft.		0.95
	33 ft. to 98 ft.		1
	> 98 ft.		<1.0
Sampling Method	Standard Sampler	C_L	1
	Sampler without liners		1.1 to 1.3

Liquefaction Evaluation and Settlement Calculation

Project Name: IV South Solar Site -- Imperial County, CA
Project No.: LE10094
Location: B-10

Maximum Credible Earthquake	7	
Design Ground Motion	0.37	g
Total Unit Weight,	115	pcf
Water Unit Weight,	62.4	pcf
Depth to Groundwater	8	ft
Hammer Effenciency	90	
Required Factor of Safety	1.0	

Boring Data						Sampling Corrections							Corrected	Fines	SPT Clean	Cyclical	Cyclical	Factor	Volumetric	Induced
Depth		Blow Counts		Liquefiable	Overburden	Sampler	SPT	Energy	Borehole	Rod	Liner	Overburden	SPT	Content	Sands	Resistance	Stress	of	Strain (%)	Subsidence
(ft)	(m)	SPT	Mod. Cal.	Soil (0 / 1)	Pressure	Diameter	N_m	C_E	C_B	C_R	C_L	C_N	$(N_1)_{60}$	%	$(N_1)_{60CS}$	$CRR_{M7.5}$	CSR	Safety		(inch)
5	1.52	16		0	575	1	16	1.5	1.0	0.75	1.1	1.70	34	90	45		0.238	Non-Liq.	0.00	0.00
10	3.05		30	0	1025	0.67	20	1.5	1.0	0.80	1	1.36	33	90	44		0.264	Non-Liq.	0.00	0.00
15	4.57		21	0	1288	0.67	14	1.5	1.0	0.85	1	1.11	20	90	29	0.369	0.312	1.41	0.00	0.00
20	6.10	17		0	1551	1	17	1.5	1.0	0.95	1.1	0.96	26	70	36		0.341	Non-Liq.	0.00	0.00
25	7.62	9		0	1814	1	9	1.5	1.0	0.95	1.1	0.86	12	70	20	0.211	0.359	0.70	0.00	0.00
30	9.14	11		0	2077	1	11	1.5	1.0	0.95	1.1	0.78	14	70	21	0.230	0.368	0.75	0.00	0.00
35	10.67	20		0	2340	1	20	1.5	1.0	1.00	1.1	0.73	24	90	34		0.368	Non-Liq.	0.00	0.00
40	12.19	72		1	2603	1	72	1.5	1.0	1.00	1.1	0.68	81	90	102		0.362	Non-Liq.	0.00	0.00
45	13.72	10		1	2866	1	10	1.5	1.0	1.00	1.1	0.64	11	80	18	0.191	0.349	0.65	1.75	1.05
50	15.24			0	3129	0.67	0	1.5	1.0	1.00	1	0.61	0	95	5	0.065	0.333	0.23	0.00	0.00
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31, 1997.

Total Settlement **1.05**

Corrections to SPT (Modified from Skempton, 1986) as listed by Robertson and Wride.

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma_{VO})^{0.5}$ C_N<=2
Energy Ratio	Donut Hammer	C_E	0.5 to 1.0
	Safety Hammer		0.7 to 1.2
	Automatic-trip Donut type Hammer		0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch	C_B	1
	6 inch		1.05
	8 inch		1.15
Rod Length	10 feet to 13 feet	C_R	0.75
	13 feet to 19.8 ft.		0.85
	19.8 ft. to 33 ft.		0.95
	33 ft. to 98 ft.		1
	> 98 ft.		<1.0
Sampling Method	Standard Sampler	C_L	1
	Sampler without liners		1.1 to 1.3

Liquefaction Evaluation and Settlement Calculation

Project Name: IV South Solar Site -- Imperial County, CA
Project No.: LE10094
Location: B-11

Maximum Credible Earthquake	7
Design Ground Motion	0.37 g
Total Unit Weight,	115 pcf
Water Unit Weight,	62.4 pcf
Depth to Groundwater	8 ft
Hammer Effenciency	90
Required Factor of Safety	1.0

Boring Data						Sampling Corrections							Corrected	Fines	SPT Clean	Cyclical	Cyclical	Factor	Volumetric	Induced
Depth		Blow Counts		Liquefiable	Overburden	Sampler	SPT	Energy	Borehole	Rod	Liner	Overburden	SPT	Content	Sands	Resistance	Stress	of	Strain (%)	Subsidence
(ft)	(m)	SPT	Mod. Cal.	Soil (0 / 1)	Pressure	Diameter	N_m	C_E	C_B	C_R	C_L	C_N	$(N_1)_{60}$	%	$(N_1)_{60CS}$	$CRR_{M7.5}$	CSR	Safety		(inch)
5	1.52		20	0	575	0.67	13	1.5	1.0	0.75	1	1.70	26	90	36		0.238	Non-Liq.	0.00	0.00
10	3.05		8	0	1025	0.67	5	1.5	1.0	0.80	1	1.36	9	90	15	0.167	0.264	0.76	0.00	0.00
15	4.57	10		0	1288	1	10	1.5	1.0	0.85	1.1	1.11	16	90	24	0.262	0.312	1.00	0.00	0.00
20	6.10	4		1	1551	1	4	1.5	1.0	0.95	1.1	0.96	6	70	12	0.132	0.341	0.46	2.30	1.38
25	7.62	8		1	1814	1	8	1.5	1.0	0.95	1.1	0.86	11	70	18	0.193	0.359	0.64	1.75	1.05
30	9.14	13		1	2077	1	13	1.5	1.0	0.95	1.1	0.78	16	70	24	0.269	0.368	0.87	1.21	0.73
35	10.67	5		0	2340	1	5	1.5	1.0	1.00	1.1	0.73	6	40	12	0.132	0.368	0.43	0.00	0.00
40	12.19	6		0	2603	1	6	1.5	1.0	1.00	1.1	0.68	7	90	13	0.141	0.362	0.47	0.00	0.00
45	13.72	10		1	2866	1	10	1.5	1.0	1.00	1.1	0.64	11	70	18	0.191	0.349	0.65	1.75	1.05
50	15.24	9		0	3129	1	9	1.5	1.0	1.00	1.1	0.61	9	95	16	0.171	0.333	0.61	0.00	0.00
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	83	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	
	0.00			0	0	0.67	0	1.5	1.0	#N/A	1	#DIV/0!	#N/A	95	#N/A	#N/A	#DIV/0!	#N/A	0.00	

Based on *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31, 1997.

Total Settlement **4.21**

Corrections to SPT (Modified from Skempton, 1986) as listed by Robertson and Wride.

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma_{VO})^{0.5}$ C_N<=2
Energy Ratio	Donut Hammer	C_E	0.5 to 1.0
	Safety Hammer		0.7 to 1.2
	Automatic-trip Donut type Hammer		0.8 to 1.3
Borehole Diameter	2.6 inch to 6 inch	C_B	1
	6 inch		1.05
	8 inch		1.15
Rod Length	10 feet to 13 feet	C_R	0.75
	13 feet to 19.8 ft.		0.85
	19.8 ft. to 33 ft.		0.95
	33 ft. to 98 ft.		1
	> 98 ft.		<1.0
Sampling Method	Standard Sampler	C_L	1
	Sampler without liners		1.1 to 1.3

APPENDIX F

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